BRIDGE CONSTRUCTION MEMO

BRIDGE CONSTRUCTION
RECORDS AND PROCEDURES

July 1, 1998

Volume II

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RALPH P. SOMMARIVA, Chief
Office of Structure Construction

BRIDGE CONSTRUCTION MEMO 100-0.0

CONCRETE MATERIALS AND MIXING

July 11, 2003

Volume II

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100-6.0	12/2/1985	CHEMICALLY REACTIVE CONCRETE AGGREGATES

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 100-1 CONCRETE MATERIALS AND MIXING**
Approved:Original Signed by Dolores Valls **D. M. VALLS, Deputy Division Chief** **Offices of Structure Construction**	**Date: July 11, 2003** **Expires: July 1, 2004** **Supersedes: BCM 100-1.0 (9/7/01)**

Subject: Freeze Thaw Requirement

The freeze-thaw resistance requirement for aggregates (CTM 528) is no longer a Standard Specification requirement. METS has determined that CTM 214, "Method of Test for the Soundness of Aggregates By Use of Sodium Sulfate", which is a current Standard Specification requirement, provides sufficient data to evaluate the aggregates' freeze thaw resistance. (Attachment 1)

All jobs advertised after June 19, 2003 should contain Amendments to the 1999 Standard Specification that eliminate the freeze thaw requirement from Section 90. A contract change order may be needed for contracts that were advertised prior to June 19, 2003. If further assistance is needed with the contract change order, please contact Agustin Perez, Division of Construction Pavement Engineer, at 916-227-5705.

c: BCR&P Manual Holders
Consultant Firms

Memorandum

Flex your power!
Be energy efficient!

To: PHILIP J. STOLARSKI
Deputy Division Chief
Materials Engineering and Testing Services
Division of Engineering Services

Date: January 29, 2003

File: Freeze-Thaw

From: TOM PYLE PJS for
Chief
Office of Rigid Pavement Materials and Structural Concrete
Materials Engineering and Testing Services
Division of Engineering Services

Subject: Freeze-Thaw Testing of Portland Cement Concrete

Over the past few years, the Office of Rigid Pavement Materials and Structural Concrete (ORPM&SC) has studied the various procedures used to evaluate aggregates for use in concrete in freeze-thaw areas. The most meaningful test found is the Sodium Sulfate Test, specifically California Test Method (CTM) 214, which is a modification of American Society for Testing and Materials (ASTM) Designation: C 88, "Soundness of Aggregates by Use of Sodium Sulfate or Magnesium Sulfate."

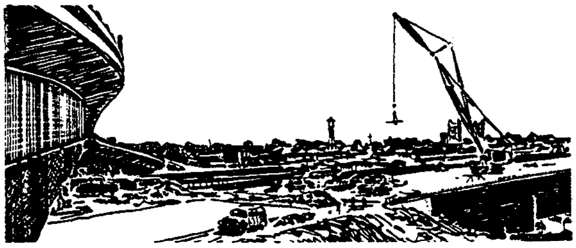
In the Scope of ASTM Test C 88, it is stated, "The internal expansive force, derived from the rehydration of the salt upon re-immersion, *simulates the expansion of water on freezing.*" Materials Engineering and Testing Services (METS) has run CTM 214 and CTM 528, "Test for Freeze-Thaw Resistance of Aggregates in Air Engrained Concrete," for over 40 years. Comparing test results from CTM 214 and CTM 528, it has been found that CTM 214 reliably corresponds to CTM 528 as to whether an aggregate will pass or fail with respect to freeze-thaw durability. The soundness test is used by at least 37 other agencies as one evaluation method, and in many agencies as the only method for determining suitability for use of aggregates in freeze-thaw areas.

CTM 214 will be implemented immediately as the method for evaluating aggregates for use in freeze-thaw areas. This implementation will require removal of all references to CTM 528 from the Standard Specifications (90-2.02), Standard Special Provision (S8-C05), and the Plans, Specifications, and Estimate Guide. This effort is being undertaken by a team composed of industry members and California Department of Transportation staff. For ongoing projects, METS will work with the Division of Construction to implement a Change Order.

Currently, about 15 to 20 state agencies use ASTM Designation: C 666, "Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing," for evaluating aggregates. The ORPM&SC now has the equipment necessary to run this test and will be evaluating the procedure to determine the potential benefits in determining the suitability of aggregates for use in freeze-thaw areas.

If you have any questions or comments or need further information, please contact me at (916) 227-7281.

c: Bob Buckley, Division of Engineering Services (DES)
Bob Pieplow, Division of Construction
Gene Mallette, Division of Construction
DES Deputy Division Chiefs
Deputy District Directors, Construction
Deputy District Directors, Engineering Services
Pavement Standards Team Office Chiefs
District Materials Engineers
John Gage, Jr., ORPM&SC
Hector Romero, ORPM&SC
Ken Beede, ORPM&SC
Doran Glauz, ORPM&SC
Bruce Carter, Hanson Aggregates and Permanente Cement



CONTROL OF CEMENT CONTENT IN CONCRETE

The Standard Specifications (Section 904.01) require that the cement content of concrete mixtures be verified in accordance with procedures described in California Test 518. Although California Test 518 is entitled "Method-of Test for Unit Weight of Fresh Concrete", it also gives instructions for determining the volume of concrete(s) per batch, and for determining the cement content (cc) in pounds per cubic yard of concrete produced.

Form' DS-OS C68, "Worksheet for California Test 518, Unit Weight of Fresh Concrete", was developed to facilitate making the calculations involved in determining that the cement content complies with specification requirements. (Attachments No. 1 and No. 2 of this Bridge Construction Memo are samples of completed Forms DS-OS C68.)

The following discussion relates to the principles underlying the unit-weight test for determining the cement content for a given concrete mixture, and for making use of Form DS-OS C68 in connection with determining the cement content.

For concrete mixes, the goal is to obtain the required cement content in each cubic yard of concrete. If it were practical to accurately weigh all the ingredients designed to produce a cubic yard of concrete, mix them thoroughly and place them in an accurate cubic yard measure, determining if the mix contained the proper amount of cement would be relatively easy, If the batch which contained the specified amount of cement overflowed the cubic yard measure, it would actually not contain enough cement per cubic yard. If the batch did not fill the cubic yard measure, it would actually contain more cement than required per cubic yard. In the event that the mix contained too much or too little cement per cubic yard, adjustments would have to be made to more nearly produce an exact cubic yard containing the specified amount of cement per cubic yard. Since it is not practical to check the cement content by making use of a cubic yard measure, the unit-weight test is used to provide the necessary data needed to make adjustments to the mix design and to make corresponding adjustments to the load weight.

In effect, these two procedures are the same except that in the unit-weight test, a small sample that is practical to handle is used and the volume produced per load is calculated by simple proportion. The unit-weight test is limited to the determination of the unit-weight of fresh concrete in pounds per cubic foot, but does include equations that may be used for calculating the volume of concrete per load and the actual cement content of the concrete produced.

In reviewing the calculations briefly, it is noted that only two factors are needed to calculate the volume of the load. The calculation for volume of load is as shown below:

W = Unit weight in pounds per cubic foot (the net weight of concrete in the calibrated bucket times the calibration factor)

Wt = The total scale weight of all the ingredients in the load of concrete

S = Volume of concrete produced per load in cubic feet

Then $S = \frac{Wt}{W}$

In other words, by simple proportion the total weight in pounds-per-load of concrete, divided by its unit weight in pounds per-cubic-foot, equals the quantity in the load in cubic feet. The important thing here is that you must have the actual weights of the water, cement and aggregates going into the load.

It is important to note that the unit-weight test does not check batching accuracy. The engineer or inspector must first be assured that scales are accurate by State Bureau of Weights and Measures inspection and seal. Batching accuracy can then be checked by observing the batching operation and by requiring the contractor to determine the gross and tare weights on the mixer truck. <u>The gross minus tare weight method should only be used as a rough check of batching accuracy, not as the value to be used in the calculations of batch volume.</u> (See Section 90-5. of the Standard Specifications.)

To determine the cement content of the concrete being produced we also need to know the actual weight of cement included in the batch. The weight of cement should be verified by observation of the scale weights as the batch is being produced. The calculation for cement content is as shown below:

*CC_A = Cement content in pounds per cubic yard

W = Number of pounds of cement in the load (verified by observation or recording equipment record

S = Volume of concrete produced per load in cubic feet (determined from unit weight as described above)

Then by proportion: $\frac{CC_A}{27} = \frac{Wa}{S}$ or $CC_A = \frac{27Wa}{S}$

*Note: CC_A = the actual weight of cement plus mineral admixture.

That is, the cement content in pounds per cubic-yard is to 27 cubic feet per cubic yard as the number of sounds in the load is to the number of-cubic feet in the load. Here it is seen that the number of pounds of cement plus mineral admixture in the load must be the actual amount of cement plus mineral admixturebatched, as determined by plant scales. Weighing is the only way to know exactly Sow much cement plus mineral admixture is in the load.

When the unit-weight test is to be performed, the actual batching of the load to be checked should be-observed and Scale weights recorded for use in determination of load volume. The mixer drum should also be checked prior to batching to be sure that a significant quantity of water is not left in the drum and unaccounted for in batch weights. It is again pointed out that if you have not verified by observation or have not been assured by automatic recording batching equipment records that the intended amount of cement plus mineral admixture was actually patched into the mixer along with the water and aggregate, the subject test and subsequent calculations cannot be used to determine the cement content.

DEPARTMENT OF TRANSPORTATION
WORKSHEET FOR CALIFORNIA TEST 518
UNIT WEIGHT OF FRESH CONCRETE
DS OS C58 (4/86)

JOB STAMP

TEST NO. ______ TEST BY ______ DATE ______

MIX NO. ______ POUR NO. ______ DEL. SLIP NO. ______ SIZE OF LOAD 7 CY

F = CALIBRATION FACTOR FOR MEASURE: 2.006

1. GROSS WT. OF MEASURE, CONCRETE AND COVERPLATE: 101.00 lb.

2. WT. OF MEASURE AND COVER PLATE: 24.25 lb.

3. NET WT. OF CONCRETE SAMPLE (line 1 minus line 2): 76.75 lb.

W = UNIT WT. OF CONCRETE SAMPLE (line 3 times F): 153.96 lb/ft^3

CC_R = CEMENT CONTENT REQUIRED BY SPECIFICATIONS (CEMENT + MINERAL ADMIX): 564 lb/yd^3

PERCENTAGE OF CEMENT REPLACED WITH MINERAL ADMIXTURE: --- %

M_R = MINIMUM WT. OF MINERAL ADMIXTURE REQUIRED: --- lb/yd^3

C_R = MINIMUM WT. OF CEMENT REQUIRED: 564 lb/yd^3

W_x = TOTAL WT. OF MINERAL ADMIXTURE PER LOAD, AS BATCHED: ----- lb

W_s = TOTAL WT. OF CEMENT PER LOAD, AS BATCHED: 3,948 lb

W_{f1} = TOTAL WT. OF FINE AGGREGATE #1 PER LOAD, INCL MOISTURE, AS BATCHED: 8,688 lb

W_{f2} = TOTAL WT. OF FINE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED: ----- lb

W_{c1} = TOTAL WT. OF COARSE AGG. #1 PER LOAD, INCL MOISTURE, AS BATCHED: 14,840 lb

W_{c2} = TOTAL WT. OF COARSE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED: ----- lb

W_{w1} = TOTAL WT. OF WATER PER LOAD AS ADDED AT PLANT (8.33 LBS PER GAL): 1,629 lb

W_{w2} = TOTAL WT. OF WATER PER LOAD AS ADDED AT JOB SITE (8.33 LBS PER GAL): ----- lb

W_t = TOTAL WT. OF ALL INGREDIENTS IN THE LOAD AS BATCHED

$= W_x + W_s + W_{f1} + W_{f2} + W_{c1} + W_{c2} + W_{w1} + W_{w2} =$ 29,105 lb

S = VOLUME OF CONCRETE PER LOAD IN CUBIC FEET $= \frac{W_t}{W} =$ 189.04 ft^3

CY = VOLUME OF CONCRETE PER LOAD IN CUBIC YARD $= S/27 =$ 7.00 yd^3

M_A = POUNDS OF MINERAL ADMIX PER CUBIC YARD OF CONCRETE PRODUCED $= \frac{W_x}{CY} =$ ----- lb/yd^3

C_A = POUNDS OF CEMENT PER CUBIC YARD OF CONCRETE PRODUCED $= \frac{W_s}{CY} =$ 564 lb/yd^3

CC_A = CEMENT CONTENT IN POUNDS PER CUBIC YARD OF CONCRETE PRODUCED $= M_A + C_A =$ 564 lb/yd^3

NOTE: Since cement content of the concrete produced equals the minimum required cement content (564 lbs), no adjustment of the mix is necessary.

FILE CATEGORY 07 ______

DEPARTMENT OF TRANSPORTATION

WORKSHEET FOR CALIFORNIA TEST 518

UNIT WEIGHT OF FRESH CONCRETE

DS CS C88 (4/86)

JOB STAMP

TEST NO. ______ TEST BY ______ DATE ______

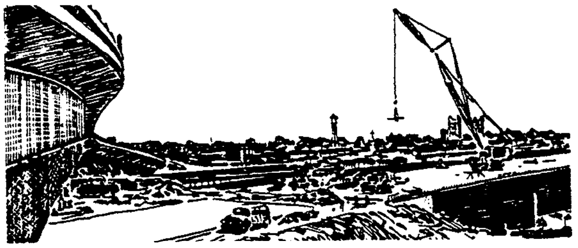
MIX NO. ______ POUR NO. ______ DEL SLIP NO. ______ SIZE OF LOAD 7 CY

F = CALIBRATION FACTOR FOR MEASURE	2.006	
1. GROSS WT. OF MEASURE, CONCRETE AND COVERPLATE	98.80	lb.
2. WT. OF MEASURE AND COVER PLATE	24.25	lb.
3. NET WT. OF CONCRETE SAMPLE (line 1 minus line 2)	74.55	lb.
W = UNIT WT. OF CONCRETE SAMPLE (line 3 times F)	149.55	lb./ft³
CC_R = CEMENT CONTENT REQUIRED BY SPECIFICATIONS (CEMENT + MINERAL ADMIX)	564	lb/yd³
PERCENTAGE OF CEMENT REPLACED WITH MINERAL ADMIXTURE	15	%
M_R = MINIMUM WT. OF MINERAL ADMIXTURE REQUIRED	84.6	lb/yd³
C_R = MINIMUM WT. OF CEMENT REQUIRED	479.4	lb/yd³
W_x = TOTAL WT. OF MINERAL ADMIXTURE PER LOAD, AS BATCHED	592	lb
W_3 = TOTAL WT. OF CEMENT PER LOAD, AS BATCHED	3,356	lb
W_{f1} = TOTAL WT. OF FINE AGGREGATE #1 PER LOAD, INCL MOISTURE, AS BATCHED	8,688	lb
W_{f2} = TOTAL WT. OF FINE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED	----	lb
W_{c1} = TOTAL WT. OF COARSE AGG. #1 PER LOAD, INCL MOISTURE, AS BATCHED	14,840	lb
W_{c2} = TOTAL WT. OF COARSE AGG. #2 PER LOAD, INCL MOISTURE, AS BATCHED	----	lb
W_{w1} = TOTAL WT. OF WATER PER LOAD AS ADDED AT PLANT (8.33 LBS PER GAL)	1,629	lb
W_{w2} = TOTAL WT. OF WATER PER LOAD AS ADDED AT JOB SITE (8.33 LBS PER GAL)	----	lb
W_1 = TOTAL WT. OF ALL INGREDIENTS IN THE LOAD AS BATCHED		
$= W_x + W_3 + W_{f1} + W_{f2} + W_{c1} + W_{c2} + W_{w1} + W_{w2} =$	29,105	lb
S = VOLUME OF CONCRETE PER LOAD IN CUBIC FEET $= \frac{W_1}{W} =$	194.62	ft³
CY = VOLUME OF CONCRETE PER LOAD IN CUBIC YARD $= S/27 =$	7.21	yd³
M_A = POUNDS OF MINERAL ADMIX. PER CUBIC YARD OF CONCRETE PRODUCED $= \frac{W_x}{CY} =$	82	lb/yd³
C_A = POUNDS OF CEMENT PER CUBIC YARD OF CONCRETE PRODUCED $= \frac{W_3}{CY} =$	465	lb/yd³
CC_A = CEMENT CONTENT IN POUNDS PER CUBIC YARD OF CONCRETE PRODUCED $= M_A + C_A =$	547	lb/yd³

NOTE: Since the cement plus mineral admixture content of the concrete produced is less than the minimum required cement content (564 lb), the weights of aggregates and water in the mix must be adjusted (reduced).

BRIDGE CONSTRUCTION MEMO 100-2.0, ATTACHMENT NO. 2
(4-30-86), Sheet 1 of 1

FILE CATEGORY 37 ______



TRANSIT-MIXED CONCRETE

In order to insure that batching and mixing of concrete which is placed in the permanent structures complies with the contract specifications, the following instructions covering inspections and documentation are to be followed.

Batch Plant Inspection

Each batch plant which furnished concrete to the project must be inspected for full compliance with the specifications. Document the inspection on Form DH-OS C54, or a similar district form. At least one plant inspection report must be in the project files before a significant portion of the work is done. Inspection reports are interchangeable between projects.

Batching and Mixing

Check the procedure for batching, charging mixers, mixing, delivery and discharge to insure that properly batched and mixed concrete is placed. This checking should be done at the beginning of the job and as often thereafter as conditions warrant. Document the checking by a separate diary covering the day or days on which it was done.

At least once during each concrete placing operation check the transit mix truck revolution count to verify proper mixing. Document the check(s) by an entry in the "Remarks" block of "Field Record for Concrete Pours" Form DH-OS C72 or the "Concrete Pour Record" Form DH-OS C73. Record the numbers of the trucks checked, time and results of check. In case of non-compliance indicate the action taken.

Load Tickets

At least once during each concrete placing operation check the load ticket for conformance with specification requirements. The checking of the tickets must be done at the time the truck arrives at the job site. Document this checking by indicating on the ticket that it has been checked, date, time, mixing revolution count and signature of the inspector.

Checking of Batch Weights

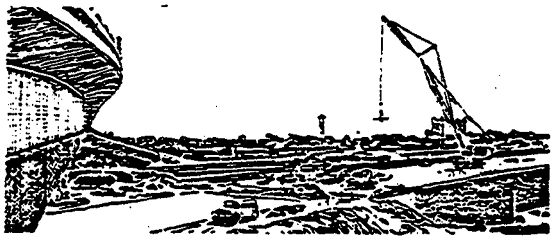
As provided in Section 90-5.03 of the Standard Specifications, the accuracy of batch weights shall be checked periodically by weighing a loaded transit mixer on platform scales, and after discharge weighing the empty truck to determine the tare and calculate the weight of the total batch. This weight should be compared with the weight of the materials placed in the truck at the batch plant. Corrective measures will be taken if the two weights are not in close agreement. This checking shall be done in conjunction with a unit weight test. The frequency of this check will depend on local conditions, but at least one check must be made every six months.

Since checks of this type are for the purpose of checking the accuracy of scales of a particular batch plant, the result of a check may be used for more than one project. Proper documentations must be in the files of each project concerned. In remote areas where there are no platform scales readily available, other means of checking the batch weights, or waiving of the check, may be authorized by the Structure Representative or the Bridge Construction Engineer. When different checking methods are used, or the checking is waived, this fact should be documented in the project records.

Variations in the prescribed procedure, to comply with local district policy, or to avoid duplication of effort, are authorized provided the extent of checking and documentation are not adversely affected.

Rejected Transit-Mixed Concrete

When it is found necessary to reject transit-mixed concrete because it is improperly mixed, has excessive slump, is over-age, etc., the necessary steps must be taken to ascertain that the rejected concrete is not used elsewhere on the contract from which it was rejected, nor on any other adjacent State contracts.



BRIDGE CONSTRUCTION MEMO 100-4.0

CONCRETE MATERIALS AND MIXING

November 2, 1982

Sheet 1 of 1

Volume II

ADMIXTURES FOR PORTLAND CEMENT CONCRETE

Admixtures for Portland cement concrete shall be used when specified or ordered by the Engineer and may be used at the Contractor'S option as provided by the Specifications.

No admixture brand shall be used in the work unless it is on list of approved brands for the type of admixture involved.

Attachment #1 of this Bridge Construction Memo gives the "Current List of Admixtures for Concrete," including explanatory information.

Attachment #2 of this Bridge Construction Memo gives information concerning the characteristics and limitations of Air Entrainers, Water Reducers, Set Retarders and combined Water 'Reducers - Set Retarders.

California Department of Transportation
Engineering Service Center
Division of Materials Engineering and Testing Services
5900 Folsom Blvd.
Sacramento, CA 95819-4612

June 2000

APPROVED ADMIXTURES FOR USE IN CONCRETE

The list of Approved Admixtures for Use in Concrete is published periodically for reference primarily by Caltrans field personnel and others involved in Caltrans projects.

As per State of California, Department of Transportation, Standard Specifications (July 1995), Section 90-4.03, no admixture brand will be used in the work unless it is on Caltrans current list of approved brands for the type of admixture involved. Admixture brands will be considered for addition to the approved list if the manufacturer of the admixture submits to the Transportation Laboratory, 5900 Folsom Blvd., Sacramento, CA 95819, a sample of the admixture accompanied by certified test results which verify that the admixture complies with the requirements in the appropriate ASTM designation. The sample shall be sufficient to permit performance of all required tests. Approval of admixture brands will be dependent upon a determination as to compliance with the specifications, based on the certified test results submitted, together with any tests Caltrans may elect to perform.

The Approved List includes only those admixtures that comply with the following ASTM designations:

C494	- Standard Specification for Chemical Admixtures for Concrete.	pp. 3 - 6
C260	- Standard Specification for Air-Entraining Admixtures for Concrete.	pp. 7 - 8
D98	- Standard Specification for Calcium Chloride.	p. 9
C618	- Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for use as a Mineral Admixture in Portland Cement Concrete	p. 10

The list provides certain essential data for field reference as well as general information that may assist in assessing properties of the plastic concrete.

The information contained herein may not be used for advertising purposes nor is it to be considered as an endorsement by Caltrans.

From ACI 212.1R, "Admixtures for Concrete"

5.2 - COMPOSITION

The materials that are generally available for use as water-reducing admixtures and set-controlling admixtures fall into five general classes:

1. Lignosulfonic acids and their salts
2 Modifications and derivatives of lignosulfonic acids and their salts
3. Hydroxylated carboxylic acids and their salts
4. Modifications and derivatives of hydroxylated carboxylic acids and their salts
5. Other materials, which include:

(i) inorganic materials, such as zinc salts, borates, phosphates, chlorides,
(ii) amines and their derivatives,
(iii) carbohydrates, polysaccharides, and sugar acids,
(iv) certain polymeric compounds, such as cellulose ethers, melamine derivatives, naphthalene derivatives, silicones, and sulfonated hydrocarbons.

These admixtures can be used either alone or in combination with other organic or inorganic, active or essentially inert substances.

NOTES:

* Chemical admixtures containing chlorides as Cl in excess of one percent by weight of admixture shall not be used in prestressed or reinforced concrete.

** When the Contractor is permitted to reduce cement content by adding chemical admixtures, the dosage of admixture shall be the dosage used in ASTM Designation: C494 for qualifying the admixtures.

† This admixture contains more than 1% chlorides as determined by California Test 415 and shall not be used in prestressed or reinforced concrete.

AE = Air Entrained
NAE = Non-Air Entrained

Type A - Water-reducing admixtures
Type B - Retarding admixtures
Type C - Accelerating admixtures
Type D - Water-reducing and retarding admixtures
Type E - Water-reducing and accelerating admixtures
Type F - Water-reducing, high range admixtures
Type G - Water-reducing, high range and retarding admixtures

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates Used to Qualify for Appropriate ASTM Tests**, fl. oz. per 100 lbs. cement (report date)	At the Qualifying ASTM Dosage(s), What Changes are Expected Relative to the Reference Concrete? Water Reduction, %	Change in AEA Dose Needed to Maintain Air Content	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer fl. oz. per 100 lbs. of cement	See Pg 2

W. R. Grace and Company
7237 East Gage Ave.
Los Angeles, CA 90040

Revised December 30, 1999

ADVA Cast-1	F	Carboxylated Polyether	<1	6.3 (1997)	AE 15.3	More	AE 1.4	3.0 to 12.0	
ADVA Flow	F	Carboxylated Polyether	<1	6.0 (1995)	AE 12.8	More	AE 1.1	3.0 to 12.0	
ADVA 100	F	Carboxylated Polyether	<1	5.2 (1999)	15.5	More	AE 0.0	3.0 to 10.0	
Daraccel	E	Calcium Chloride, Triethanolamine	>20	16.0 (1983)	AE 5.7	Less	AE (1.4)	8.0 to 40.0	†
Daracem 50	A	Lignin, Calcium Chloride, and Polymers	>8	5.0 (1992)	AE 7.6	Less	Negligible	5.0 to 7.0	†
Daracem 55	A	Lignin, Calcium/Sodium Nitrate, Polymer	<1	4.0 (1992)	AE 5.8	Less	AE 0.9	3.0 to 9.0	
Daracem 100	A, F	Naphthalene Sulfonate	<1	8 (1991)	AE 11.5	Less	AE 0.3	9.0 to 11.0	
Daracem 100	G	Naphthalene Sulfonate	<1	12 (1991)	AE 15.0	Less	AE 3.3	12.0 to 15.0	
Daracem ML 330	F	Melamine-Formaldehyde Polymer	<1	14.5 (1998)	AE 15.4	More	AE 1.2	6.0 to 25.0	
Daracem ML 500	A, F	Melamine and Naphthalene Sulfonate Formaldehyde Co-Polymers	<1	12.0 (1999)	AE 12.0	More	AE 0.4	6.0 to 25.0	
Daratard 17	B, D	Hydroxylated Organic Compounds	<1	3.0 (1992)	AE 8	More	AE 2.0	2.0 to 7.0	
Darex Corrosion Inhibitor	C	Calcium Nitrite Aqueous Solution	<1	78.0 (1979)	Negligible	Same	AE (2.0)	50.0 to 170.0	
Daracem 19 (formerly WRDA 19)	A, F	Naphthalene-Sulfonate Formaldehyde Copolymer	<1	8.0 to 25.0 (1981)	AE 20 to 30	Less	AE 0.5 to 1.0	8.0 to 25.0	
Daraset	C	Calcium Nitrate Solution	<1	45.0 (1994)	AE 4	Same	A (1.0)	20.0 to 50.0	
Polarset	C	Calcium Nitrate/ Nitrite Solution	<1	30.0 (1994)	AE 5	Same	A (3.0)	8.0 to 100.0	
WRDA 20	A	Glucose Polymers, Lignosulfonate, and Amine	<1	2.5 (1985)	AE 6.8	Less	AE 1.0	2.5	
WRDA 64	A	Lignosulfonate, Amine, and Glucose Polymer	<1	3.0 (1979)	AE 11	Less	AE 1.4	3.0 to 5.0	

					At the Qualifying ASTM Dosage(s), What Changes are Expected Relative to the Reference Concrete?			Dosage Rate Suggested by Manufacturer	
Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates Used to Qualify for Appropriate ASTM Tests**, fl. oz. per 100 lbs. cement (report date)	Water Reduction, %	Change in AEA Dose Needed to Maintain Air Content	Initial Set Retardation, (Acceleration) hours	fl. oz. per 100 lbs. of cement	See Pg 2
WRDA 79	A, D	Modified Lignosulfonate	<1	5.0 to 7.5 (1980)	AE 8 to 10	Less	AE 1.0 to 2.2	4.0 to 10.0	
WRDA 82	A	Lignosulfonate and Amine	<1	3.0 (1983)	AE 6.1	Less	AE 0.2	3.0	
WRDA w/Hycol	A	Organic Compounds w/Hydration Control Agent	<1	3.0 and 5.0 (1974)	AE 5 to 7	Less	AE (0.3) to 1.3	3.0 to 5.0	
Recover	D	Hydroxycarboxylic Acid Salts	<1	5.0 (1992)	AE 9.0	Same	AE 1.7	2.0 to 16.0	

Hill Brothers Chemical Co.
1675 N. Main Street
Orange, CA 92667

HICO 610	A	Sodium Lignosulfonate	<1	5.0 (1987)	NAE 5.7	Not Tested for Air Entrained Concrete	NAE (1)	5.0 to 12	
HICO 911	C	Polymer Modified Calcium Chloride	>33	24 (1992)	NAE 2.7	Not Tested for Air Entrained Concrete	NAE (2.0)	32 to 64	†

Master Builders
23700 Chagrin Blvd.
Cleveland, OH 44122

Revised December 30, 1999

Pozzolith 400-N	A, F	Naphthalene Sulfonate	<1	15.0 (1991)	AE 28	More	AE 0.2	10.0 to 20.0	
Delvo Stabilizer	B, D	Salts of Organic Agent	<1	4.0 (1992)	AE 7.8	Less	AE 1.1	2.0 to 130	
Master Pave N	A	Glucose Polymer	<1	2.0 (1989)	AE 6.0	Less	AE 0.4	2.0 to 4.0	
MBL-82	A	Lignin	<1	5.0 (1991)	AE 7	Less	AE 0.3	5.0 to 10.0	
MBL-82	B, D	Lignin	<1	8.0 (1990)	AE 9.5	Less	AE 1.6	5.0 to 10.0	
Polyheed	A	Lignosulfonate, Triethanolamine, Ammonium Thiocyanate	<1	7.0 (1991)	AE 6.1	Same	AE 0.4	3.0 to 12.0	
Polyheed RI	B, D	Cement Dispersing Agent	<1	4.0 (1994)	AE 7.6	Less	AE 1.25	3.0 to 12.0	
Polyheed FC 100	A,C,E ,	Cement Dispersing Agent	<1	9.0, 15.0 (1998)	AE 6.7	More	AE (0.7)	8.0 to 30.0	
Pozzolith NC 534	C	Cement Dispersing Agent	<1	27.0 (1993)	AE 5.7	More	AE (1.7)	10.0 to 45.0	

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates Used to Qualify for Appropriate ASTM Tests**, fl. oz. per 100 lbs. cement (report date)	At the Qualifying ASTM Dosage(s), What Changes are Expected Relative to the Reference Concrete? Water Reduction, %	Change in AEA Dose Needed to Maintain Air Content	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer fl. oz. per 100 lbs. of cement	See Pg 2
Polyheed 997	A	Lignosulfonate Triethanolamine	<1	5.0 (1990)	AE 6.9	Less	AE 0.4	3.0 to 12.0	
Polyheed 997	F	Lignosulfonate Triethanolamine	<1	8.0 (1990)	AE 12.3	Less	AE 0.3	3.0 to 12.0	
Pozzolith 100-XR	B, D	Glucose Polymer	<1	2.5 (1993)	AE 6.1	Same	AE 1.5	2.0 to 4.0	
Pozzolith 122 HE	C, E	Cement Dispersing Agent	>24	17.0 (1998)	AE 5.5	More	AE (1.1)	16.0 to 64.0	
Pozzolith 200 N	A,B,D	Cement Dispersing Agent	<1	4.0 (1998)	AE 6.9	Less	AE 0.7	3.0 to 5.0	
Pozzolith 220 N	B, D	Polymer, Triethanolamine	<1	3.5 (1991)	AE 5.8	Less	AE 1.8	2.0 to 5.0	
Pozzolith 220-N	A	Polymer, Triethanolamine	<1	2.0 (1991)	AE 6.2	Less	AE 0.5	2.0 to 5.0	
Pozzolith 300 N	A	Polymer, Triethanolamine	<1	3.0 (1990)	AE 7-8	Less	AE 0.3	3.0 to 5.0	
Pozzolith 300-R	B, D	Polymer	<1	5.0 (1990)	AE 10	Less	AE 2.6	3.0 to 5.0	
Pozzolith 322-N	A	Polymer, Triethanolamine	<1	4.0 (1990)	AE 8.0	Less	AE 0.7	3.0 to 7.0	
Pozzolith 344-N	A	Calcium Chloride, Triethanolamine	>8	6.0 (1991)	AE 6.5	Less	AE 0.6	3.0 to 9.0	†
Pozzutec 20	C, E	Polymer	<1	15.0 (1990)	AE 5.5	More	AE 1.1	5.0 to 90.0	
Rheobuild 1000	A, F	Naphthalene Sulfonate	<1	15.0 (1988)	AE 18	Less	AE 0.4	5.0 to 25.0	
Rheobuild 2000 B	A, F	Cement Dispersing Agent	<1	10.0 (1994)	AE 13.9	More	AE 1.1	10.0 to 25.0	
Rheobuild 3000 FC	A, F	Based on Glenium Technology	<1	4.0 (1998)	AE 12.4	Less	AE 0.2	4.0 to 12.0	
RMC 121	A	Lignosulfonate Triethanolamine	<1	5.0 (1990)	AE 6.9	Less	AE 0.4	3.0 to 12.0	
RMC 121	F	Lignosulfonate Triethanolamine	<1	8.0 (1990)	AE 12.3	Less	AE 0.3	3.0 to 12.0	
Pozzolith 80	A, B, D	Cement Dispersing Agent	<1	3.0 (1998)	AE 6.8	Same	AE 0.2	4.0 to 10.0	

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates Used to Qualify for Appropriate ASTM Tests**, fl. oz. per 100 lbs. cement (report date)	At the Qualifying ASTM Dosage(s), What Changes are Expected Relative to the Reference Concrete? Water Reduction, %	Change in AEA Dose Needed to Maintain Air Content	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer fl. oz. per 100 lbs. of cement	See Pg 2

Sika Chemical Corporation
1372 East 15th Street
Los Angeles, CA 90021

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates (report date)	Water Reduction, %	Change in AEA Dose	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer	See Pg 2
Plastocrete 161	A	Lignosulfate	<1	4 (1982)	AE 7.7	Same	AE 0.2	3.0 to 5.0	
Plastocrete 161 FL	C	Inorganic Salt-Organic Mixture	<1	16 (1987)	AE 5.4	Same	AE 1.25	12.0 to 24.0	
Plastocrete 161 HE	C	Calcium Chloride Triethylamine	>5	34 (1978)	AE 1.3	Less	AE (1.0)	6.0 to 64.0	†
Plastocrete 161 MR	B, D	Lignosulfonates	<1	2.9 (1989)	AE 7.4	Same	AE 2.4	3.0 to 6.0	
Plastocrete 169	A	Lignosulfonates	<1	4 (1985)	AE 8.73	Same	AE (0.25)	3.0 to 7.0	
Plastocrete 169	B, D	Lignosulfonates	<1	6 (1986)	AE 22	Same	AE 2.3	3.0 to 7.0	
Plastiment	B, D	Hydroxylated Carboxylic Acid	<1	4.0 (1990)	AE 7.3	Same	AE 3.1	2.0 to 4.0	
Sikament FF	F	Melamine Polymer	<1	12 (1994)	AE 12.2	Same	AE 1.3	10.6 to 21.2	
Sikament 86	F	Melamine Polymer	<1	12 (1994)	AE 14.4	Same	AE 0.7	10.6 to 21.2	
Sikament 300	F	Blend Sodium Alkylnapthalene	<1	12 (1992)	AE 12.2	Same	AE 1.0	6.0 to 24.0	
Plastiment NS	A	Lignosulfonates	<1	4 (1996)	AE 7.6	Less	AE 1.1	2.0 to 4.0	
Sika-Rapid- 1	C	RMF-1503	<1	20 (1996)	AE 3.1	Less	AE (1.6)	4.0 to 48.0	

Boral Material Technologies, Inc.
45 N. E. Loop 410, Suite 700
San Antonio, TX 78216

Revised December 30, 1999

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates (report date)	Water Reduction, %	Change in AEA Dose	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer	See Pg 2
Boral RDI	F,G,	Sulfonated Naphthalene Formaldehyde	<1	4.0 (1990)	AE 15.3	Same	AE (1.0)	6.0 to 20.0	
Boral LR	A,D	Lignosulfonate	<1	6.0 (1997)	AE 8.0	Less	AE (1.1)	4.0 to 6.0	
Boral NR	A,D	Lignosulfonate Based Material	<1	3.0 (1997)	AE 6.7	Less	AE (1.2)	3.0 to 6.0	
Boral NW	A,D	Lignosulfonate Based Material	<1	3.0 (1997)	AE 7.5	Less	AE (0.2)	3.0 to 6.0	
Boral X15	A	Lignosulfonate Based Material	<1	4.0 (1993)	AE 5.4	Less	AE (0.1)	3.0 to 10.0	
Boral ACN	C, E	Blend of Admixture	<1	60.0 (1998)	AE 8.6	More	AE (1.6)	20.0 to 50.0	

Product or Brand Name	ASTM Type	Class or Composition	Chloride Content* %	Dosage Rates Used to Qualify for Appropriate ASTM Tests**, fl. oz. per 100 lbs. cement (report date)	At the Qualifying ASTM Dosage(s), What Changes are Expected Relative to the Reference Concrete? Water Reduction, %	Change in AEA Dose Needed to Maintain Air Content	Initial Set Retardation, (Acceleration) hours	Dosage Rate Suggested by Manufacturer fl. oz. per 100 lbs. of cement	See Pg 2
Boral SP	A, F	Sulfonated Napthalene Formaldehyde Condensate	<1	7.0 (1998)	AE 17.1	More	AE (0.2)	6.0 to 25.0	
Boral HC	A, B, D	Carbohydrate Salts	<1	2.5, 5 (1998)	AE 6.5 AE 7.3	Same Same	AE 0.6 AE 2.0	2-6	
Boral TR	B, D	Carbohydrate Salts	<1	4.0 (1998)	AE 6.8	Less	AE 3.1	3-6	
Boral LW*	A	Lignin Family	<1	3.0 (1997)	AE 9.5	Less	AE 0.3	3-10	
Boral HW	A	Lignin Family	<1	6.0 (1998)	AE 6.9	Less	AE 1.0	3-10	

* Boral HW contains calcium chloride, therefore not recommended for post tension and pre-stressed concrete.

The Euclid Chemical Company
19218 Redwood Road
Cleveland, Ohio 44110-2799
Tel. No: (216) 531-9222

June 1, 2000

Accelguard HE	E	Calcium Chloride based Material	31-35	24 (1997)	AE 6.3	More	(1.5)	16-32	
Eucon 37	A, F	Napthlene Sulfonate	<1	16 (1999)	AE 18.31	Same	AE 0.7	10-16	
Eucon Retarder 100	D	Sodium Gluconate	<1	3 (1999)	AE 6.4	Less	AE 1.9	2-6	
Eucon MR	A	Calcium Nitrate & Calcium Ligno Sulfonate Material	<1	6 (1999)	AE 7.1	Same	AE 1.1	4-10	
Eucon WR	A	Calcium-Sodium Ligno Sulfate	<1	5 (1997)	AE 8.3	Less	AE 0.5	4-5	
Eucon WR-91	A	Calcium Ligno Sulfonate	<1	3 (1999)	AE 6.4	Less	AE 0.6	2-6	

Product or Branch Name	Class or Composition	Chloride Content (percent)	Report Date	Dosage Rate Suggested by Manufacturer

Master Builders
32700 Chagrin Blvd.
Cleveland, OH 44122

MBVR Standard	Vinsol Resin	<1	1991	0.4 to 4.0
MB-VR Concentrated	Vinsol Resin	<1	1992	0.4 to 4.0
MBAE-90 also called Pave Air 90	Rosin Soap	<1	1993	0.25 to 4.0
Micro-Air	Fatty acid Salts	<1	1991	1.0
Pave-Air	Vinsol Resin	<1	1992	1.0

W. R. Grace and Company
7237 East Gage Ave.
Los Angeles, CA 90040

Amex 210	Benzene Sulfonate Sodium Salt	<1	1989	0.5 to 8.0
Darex AEA	Organic Acid Salts	<1	1975	0.8
Darex II AEA	Alkaline Solution of Fatty Acid Salts	<1	1993	0.75 to 3.0
Daravair 1000	Neutralized Resin and Rosin	<1	1994	0.75 to 3.0
Daravair	Neutralized Resin and Rosin	<1	1994	0.75 to 3.0
Daravair M	Neutralized Vinsol Resin	<1	1975	1.0
Daravair AT 60	Aqueous Solution of Neutralized Vinsol Resin, Amine and Fatty Acids	<1	1994	0.5 to 3.0

Sika Chemical Corporation
1372 East 15th Street
Los Angeles, CA 90021

Sika AER	Neutralized Vinsol Resin	<1	1986	0.5 to 1.5
Sika AEA 15	Sodium Salt Type Soap	<1	1983	0.5 to 1.5
Sika AEA 14	Sodium Salt of an Organic Ester	<1	1996	0.5 to 3.0

Hill Brothers Chemical Company
1675 North Main St.
Orange, CA 92667-3442

HICO-315-L	Sodium Tall Oil Fatty Acid Soap	<1	1968	0.75 to 3.0

Product or Branch Name	Class or Composition	Chloride Content (percent)	Report Date	Dosage Rate Suggested by Manufacturer

Boral Material Technologies, Inc.
45 N. E. Loop 410, Suite 700
San Antonio, TX 78216

Boral Air 40	Resin Surfactant	<1	1997	1.0

The Euclid Chemical Company
19218 Redwood Road
Cleveland, Ohio 44110-2799

AEA – 92	Synthetic Organic Chemicals based Admixture	<1	1992	½ to 1.0

Product	Type or Composition (Solid or Solution)	Calcium Chloride Content in Percent (given for solution form only)	Grade (given for solid form only)

Hill Brothers Chemical Company
1675 North Main St.
Orange, CA 92667-3442

HB-98	Solution	30.1	

Lee Chemical, Incorporated
3113 McKinley Way
Costa Mesa, CA 92626

ASTM Grade	Solution	33.0	

Cargill
Solarchem Resources
7200 Central Avenue
Newark CA 94560-4206

Liquid Calcium Chloride, Technical Grade, Treated	Solution	38.3	

Company Name	Classification of Mineral Admixture	% Calcium Oxide (Range)
(1) Boral Materials Technology (formerlyWestern Ash Company) 7500 N Dreamy Draw, Suite 234 P.O. Box 7360 Phoenix, Arizona 85036		
(a) Navajo Fly Ash (Page, Arizona)	F	6.1 to 7.9
(b) Mojave Fly Ash (Laughlin, Nevada)	F	8.5 to 9.9
(c) Apache Fly Ash (Cochise, Arizona)	F	3.0 to 8.0
(d) Snowflake (Snowflake, Arizona)	F	3.0 to 4.2
(e) Monticello (Monticello, Texas)	F	7.1 to 8.0
(2) ISG Resources, Inc. 7525 S.E. 24th Street Mercer Island, Washington 98040		
(a) Centralia Fly Ash (Centralia, Washington)	F	7.6 to 8.0
(b) Jim Bridger Fly Ash (Rock Springs, Wyoming)	F	6.2 to 7.5
(c) IPSC/Delta Fly Ash (Delta, Utah)	F	9.1 to 9.9
(d) Hunter Fly Ash (Castle Dale, Utah)	F	7.9 to 9.9
(3) Phoenix Cement Company 2501 W. Behrend Drive P.O. Box 43740 Phoenix, AZ 85080		
(a) Cholla Fly Ash (Joseph City, Arizona)	F	3.1 to 5.0
(b) Four Corners Fly Ash (Fruitland, New Mexico	F	2.4 to 2.8
(4) Mineral Resources Technologies, LLC 120 Interstate North Parkway East, Suite 440 Atlanta, GA 30339		
(a) Coronado Fly Ash (St. John, Arizona)	F	2.6 to 5.0

ADMIXTURES
FOR
PORTLAND CEMENT CONCRETE

Admixtures are used in Portland cement concrete to improve either its placing properties or its hardened qualities, or both. It is not uncommon to use more than one admixture in a mix to achieve these ends. Since not all admixtures are compatible with each other and since some admixtures are harmful to the finished concrete, the engineer should become familiar with their purposes, characteristics and limitations so that he can use them discriminately to overcome or minimize some of the problems encountered in concrete construction and performance.

Some commonly used admixtures are:

Air Entrainers

Water Reducers

Set Retarders

Combined Water Reducers - Set Retarders

Accelerator admixtures are also available for concrete, but since they normally contain chlorides which are harmful to the reinforcing steel their use is prohibited by the Department of Transportation. Hence, the accelerator admixtures will not be discussed here.

Air Entrainment

Air entrainment of a properly designed concrete mix results in several advantages over a non air entrained mix. Among these are:

1. Increased freeze-thaw resistance of hardened concrete.

2. Improved fluidity and uniformity.

3. Reduced bleeding.

4. Increased cohesiveness and less segregation.

5. Reduced mix water requirement.

Non air entrained concrete normally contains about 1 or 2 percent air in the form of relatively large voids. This is termed entrapped air. Entrained air, on the other hand, exists as very small bubbles and in an amount that depends on the amount of the air entraining agent added to the concrete as well as on other variables of the mixture and environment. In air entrained concrete the total air content recommended for normal uses varies with the maximum size of the aggregate and ranges from about 5 percent for a l½ -in. maximum size to about 9 percent for a 3/8-in. maximum size. These are optimum values for freeze-thaw resistance. Amounts needed to effect the other potential benefits from air entrainment vary according to many factors, the most important being the objective sought. In no case should the amount of purposely entrained air be greater than that listed as required for freeze-thaw resistance. The need for varying air content with aggregate size to effect freeze-thaw resistance is because it is the mortar portion of the mix which is being protected by the entrained bubbles. Hence, more total air is needed with smaller size aggregate than with larger because more mortar is present in a mix with the smaller size.

Experience in the use of air entrained concrete has shown the necessity for some minor changes in certain construction practices. For example, air entrained concrete is more sticky than ordinary concrete, especially in rich mixes. The concrete may therefore have a tendency to tear under the strike-off of the finishing machine. This tendency can usually be corrected by increasing the frequence of the transverse oscillations of the strike-off. Sometimes a slight adjustment of the mixture composition, such as a change in the fine aggregate-coarse aggregate ratio, will accomplish the desired results.

Normally, a 1/8-in. backward tilt or lift of the cutting edge of the strike-off board and a level position of the finish float have been found best for finishing air entrained concrete. Rebound has sometimes been experienced behind the finishing machine. This is caused by too much material passing under the strike-off. The condition can be controlled by changing the tilt, as mentioned previously, or by regulating the height of the concrete roll being carried ahead of the strike-off. The concrete ahead of the strike-off during the first pass of the finishing machine should have a uniform depth of approximately 3 inches or less.

Bleeding is reduced and little water rises to the surface of air entrained concrete. Ordinarily, finishing operations can be done sooner on air entrained concrete.

If there is no alteration of the mixture proportions, air entrained concrete will have a lower strength than that of ordinary concrete, This reduction will be about 5 percent for each percent of added air. However, owing to the increased plasticity imparted by entrained air, it is usually possible to reproportion the mixture using lower water and sand contents at constant slump and thereby cause less strength reduction. This is particularly true of lean concretes or those with a small maximum size *of* aggregate.

Because any increase of the air content above the intended amount will result in further loss of strength, it is important to maintain close control of the aircontent by frequent determinations at the job site. A common error is to neglect the frequent checking of the air content.

When a separate air entraining admixture is needed in concrete containing a water reducing, or a set retarding, or a combined water reducing and set retarding admixture, the amount required to produce a given volume of air is usually less than that for concrete without the admixture.

The proportioning of air entrained concrete is similar to that of ordinary concrete. The volume of air is merely another component that is added into the summation of absolute volumes along with those of the other ingredients. A simplified rule-of-thumb method for reproportioning the concrete mix for air entrainment is as follows:

Sand: Reduce volume by 67% of the volume of entrained air.

Water: Reduce volume by 33% of the volume of entrained air.

It has been proven that a given quantity of an approved air entraining agent will produce a constant amount of air in concrete mixtures if all other conditions remain constant, Unfortunately, due to changes which take place during the concrete pour, the conditions in actual practice are constantly changing. Hence, the percent air changes with them and it is not uncommon for the percent of air in air entrained concrete to vary from batch to batch.

Inconsistencies of air in air-entrained concrete are usually of great concern to those who experience it for the first time, To help relieve this concern and to help the experienced as well as the inexperienced in maintaining better control of the percent air, conditions which would cause a <u>decrease</u> in the percent of air by volume in a concrete mix are listed:

1. Increased concrete temperature.

2. Decreased water/cement? ratio or decreased consistency. (Decrease in air by volume is approximately 0.5% for each one-inch decrease in slump below seven inches, Increase in slump over seven inches may also decrease the percent air.)

3. Increased quantity of crushed sand, (Vs. round sand)

4. Decreased quantity of crushed aggregate. (Vs. round agg.)

5. Decreased quantity of 30-50 mesh material.

6. Increased quantity of below 200 mesh material. (Cement, very fine sands, silt, etc.)

7. Decreased quantity of large aggregate. (Vs. small aggregate)

8. Charging of agent to the aggregate instead of to the water. (This practice may be responsible for large batch to batch variation.)

9. Artifically heating or cooling the aggregate or water.

10. Increased water "hardness".

11. Increased cement content.

12. High agitation speed.

13. Prolonged mixing.

14. Over loaded mixer,

15. Increased mixing blade wear. (Possibly one reason for variations in air between transit mixers.)

16. Impaired mixing action. (Hardened mortar accumulated on mixing blades.)

17. Increased vibration time (over 15 seconds).

It has been established that air entrainment in concrete increases freezing and thawing resistance, decreases segregation, decreases bleeding and improves fluidity. Proper control is, however, important to ensure maintaining the air content within desired limits --high enough to derive the benefits of durability and workability but low enough to avoid unnecessary decreases of strength.

Water Reducing and Set Retarding Agents

Water reducing, retarding, and combined water reducing and retarding admixtures are used to modify the properties of fresh and hardened concrete. These modifications include increasing the fluidity and working qualities of a given mixture without an increase in water content or decrease in strength. Retarders are especially beneficial in delaying set during hot, dry weather, or for elimination of cold joints and discontinuities in large structure units, or for elimination of stress buildup in freshly placed concrete such as occurs over supports of continuous units.

Many of the admixtures minimize segregation of the mix during placing. This is an important attribute in preventing rock pockets and grout leaks.

It should be noted that all cement brands of a given type may not be compatible with a given admixture, Hence, it is cautioned that the experience gained on one job with a given admixture cannot necessarily be used without question on another which uses a different brand of cement.

The most commonly used water reducing retarders are grouped into two general categories: Lignosulfonates (LS) and Hydroxylated Carboxylic Acids (HC).

Generally the lignins (LS) increase drying shrinkage, entrain some air and reduce bleeding, The hydroxyls (HC) usually have little effect on shrinkage but do increase bleeding.

The effects of the admixture on the properties of fresh concrete vary with the type of admixture and the materials composing the concrete. The properties that are most affected are slump, air content, water requirement, bleeding, segregation, rate of hardening, workability, and strength.

The slump, air content, and water requirement of concrete are interrelated so that the specific effect of admixtures on each of these properties is difficult to separate, Indication of the individual effects can, however, be gained by considering the behavior when two of the variables are controlled. The cumulative effects may then be related in a rational manner.

Water reducing, set retarding admixtures modify workability of concrete through improved mobility and by lengthening the hardening period. For equal water and air contents, the slump of concrete containing an admixture with water reducing properties may be increased up to 100 percent as compared with comparable reference concrete within the range of slump ordinarily used, The change in slump increase with given increase of water content is usually more pronounced in admixtured concrete than in conventional concrete, Relatively small changes in water content in some cases may markedly affect the fluidity of the mixture. The placability of concrete at a given slump is generally improved for concretes containing these admixtures.

Some admixtures increase bleeding of the concrete. Normally bleeding is considered objectionable because it causes segregation, causes water gain under large aggregate and reinforcing steel and causes vertical water "pipes" in the concrete. However, under certain conditions, bleeding is desirable. For instance, when concrete is placed industry, hot weather, bleed water rising to the surface replaces the evaporating water and precludes the undesirable practice of adding water to facilitate finishing. Bleeding is also desirable when the concrete is to be or can be revibrated. Revibation after the bleeding has subsided lowers the water cement ratio in the finished work and also makes it more dense.

For concretes of equal slump and air content, the use of water reducing, set retarding admixtures will permit a reduction of water content up to 12 percent. The corollary properties associated with reduction in water requirement are also important, because a given water/cement ratio can be maintained while producing concrete of a higher slump, or if advantage is taken of the possible water reduction, a specific strength may be obtained with a cement content proportionately lower than in concrete not containing the admixture.

The retarding admixtures delay setting, but, after setting has occurred, the hydration reactions, hardening, and strength development proceed at normal or accelerated rates. Concrete prepared with these admixtures will normally give compressive strengths at 24 to 48 hours equal to and often at later ages higher than those of concrete of the same cement content, air content, and slump without the admixtures, The amount of strength at early ages is somewhat dependent on the amount of retardation, and for concrete in which the setting time is abnormally long, an increase may not be achieved. The increase at 28 days may be as high as 10 to 20 percent.

In general, the modulus of elasticity and bond to reinforcing steel are increased and creep of the concrete is decreased by the use of these admixtures, but the changes appear to relate to the improvement in compressive strength.

The effectiveness of water reducing admixtures varies with the water/cement ratio of the mixture, Concrete of excessive water content and high water/cement ratio cannot benefit by this type of admixture to the extent realized in a medium to low slump concrete.

The addition of these materials to the mixture in liquid form is highly desirable to obtain a more uniform distribution throughout the concrete mass within the time required to mix the concrete. Care should be taken when using liquids to avoid adding them directly to the cement or dry, absorptive aggregate. A fixed procedure for the method and time of dispensing the admixture should be followed for each job.

The time of addition of set retarding admixture may have a marked effect on the resulting mixture. A delay of l/2 to 2 min. in adding the admixture after all other materials are batched and mixing started will often result in an increase in slump and retardation.

Water reducing and set retarding admixtures are generally used in relatively small quantities. It is therefore important that suitable and accurate dispensing equipment be used. These admixtures are available as both liquids and powders. Water-soluble powders should be dissolved in water prior to use so that they can be dispensed as liquids. Unless complete solution of the admixture is obtained, the solutions should be agitated before being taken from storage or shipping tanks or containers, and agitation should be provided before and during dispensing into the concrete mixture. Manufacturers' recommendations should be followed.

Powdered admixtures that are not completely water soluble may be manually dispensed dry by volume or weight. In this case, they should preferably be added to the fine aggregate.

General

There are innumerable products proposed for concrete admixtures. Unfortunately, a number of these are being promoted without supporting data by salesmen without technical background. Admixtures are tested by the Transportation Laboratory to determine the effects of an admixture on shrinkage, setting time and compressive strength. The T.L. test report on an admixture lists the recommended maximum dosages for specific work, Although retardation tests are made in the Laboratory, the results may not be directly applicable to field application due to the environmental difference.

Should it be desirable to add both an air entraining and water reducing retarder admixture to a mix, their compatibility should be predetermined. Some admixture combinations react chemically to form a jelly-like substance, which will more than likely nullify or severely reduce the effectiveness of either admixture and might in fact produce a detrimental effect.

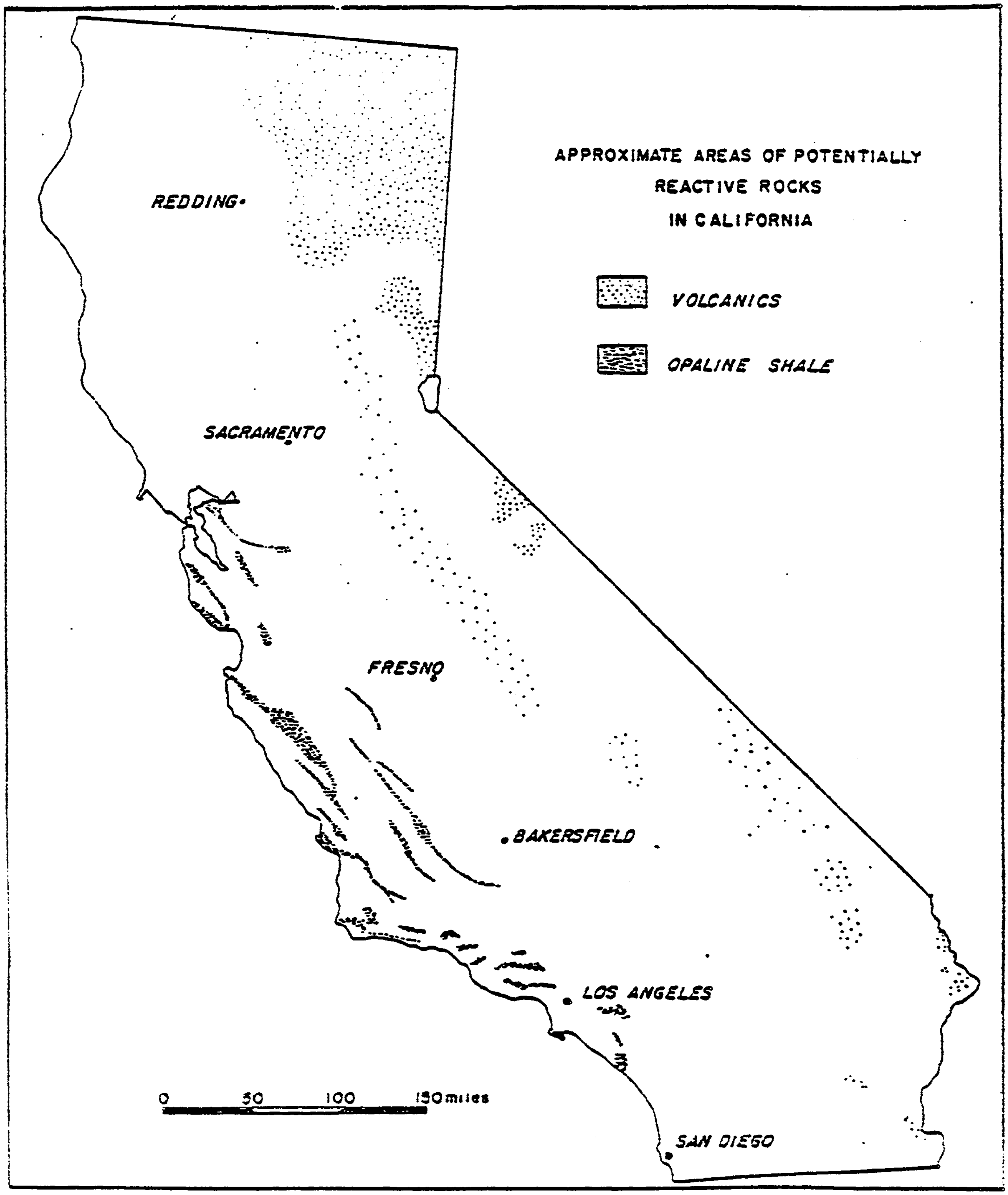
The Transportation Laboratory will provide background information, advice and field assistance on the use of any admixture upon request.

CHEMICALLY REACTIVE CONCRETE AGGREGATES

Some California concrete aggregates are known to be chemically reactive with "Type II Modified" cement.

Attachment No. 1 to this Bridge Construction Memo is a map of California showing approximate distribution of potentially reactive rocks.

Structure Representatives should check to determine if the aggregate for concrete on their projects comes from one of the areas of potentially reactive rocks. If it appears that concrete aggregates do come from one of these areas, the concrete section of the Transportation Laboratory should be contacted to determine if there is a problem with using the aggregate, and to determine what steps the Laboratory recommends to mitigate the problem.



Map of California showing approximate distribution of potentially reactive rocks.

Volume II

BRIDGE CONSTRUCTION MEMO 105-0.0

CONCRETE – PLACE, FINISH & PROTECT

October 19, 1998

Sheet 1 of 1

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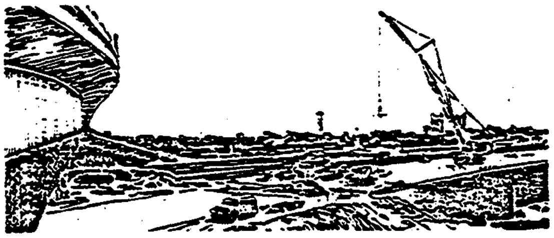
105-5.0	12-02-96	USE OF “BURLENE” FOR WATER CURE
105-6.0	12-02-96	USE OF “VISQUEEN” FOR COLUMN CURE

RALPH P. SOMMARIVA, Chief
Office of Structure Construction



MISCELLANEOUS FINISHING INFORMATION

WHIP-BLAST CONCRETE SURFACE FINISH

"WHIP-BLASTING" may be used to produce a surface finish which conforms to the Specification requirements for a Class 1 Surface Finish, providing some precautions are taken. "Whip-Blasting" generally gives best results when applied to concrete which has a hard surface. This condition generally occurs when the concrete surfaces have been formed with steel, fiber-glass or coated high density plywood forms. In order to obtain a good appearance, it is necessary to do some experimenting to determine proper grain size, air pressure, distance of sand blast nozzle from concrete surface, angle of application of sand, etc. Once these factors have been determined, the Bridge Representative must inspect the work periodically to be assured that workmen are following those procedures which it has been determined will give satisfactory results.

FINISHING STRUCTURES OVER TRAFFIC

When it is necessary that the Contractor perform finishing operations over a travelled way, lane closures should be made beneath the finishing operation if there is any possibility that the scaffolding will be lower than the bridge soffit, or if there is any possibility that objects will fail to the travelled way.

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 105-1.1** **Concrete – Place, Finish, and Protect**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 105-1.1 Dated July 1, 1998**

Subject: Glue Additives For Concrete Patching Mortar

Patch material used for achieving an ordinary surface finish per Section 51-1.18A of the Standard Specifications shall conform to the requirements of Section 51-1.135 "Mortar" of the Standard Specifications. Additives to the mortar are acceptable provided they meet the following requirements:

1) The additive must not have polyvinyl acetate as the active ingredient.

2) The additive must be acrylic based.

For questions on whether a particular glue additive is acceptable contact the Headquarters Office of Structure Construction or your regional Structural Materials Representative (SMR). Use the following link to determine the appropriate SMR to contact: http://www.dot.ca.gov/hq/esc/Translab/smforms/StructuralMaterialsRepresentatives.doc

c: BCR&P Manual Holders
Consultant Firms
R.Pieplow, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN **Approved:**____________________ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 105-1.2** **CONCRETE – PLACE, FINISH, AND PROTECT** **Date: October 1, 1998** **Expires: November 1, 1999** **Supersedes: none**

Subject: Acceptable Class 1 Surface Finishes

A Class 1 Surface Finish shall conform to the provisions of Section 51-1.18B of the Standard Specifications. A Class 1 Surface Finish is a smooth, even surface of uniform texture and appearance, free of unsightly bulges, depressions, and other surface imperfections.

The amount of effort required to produce a Class 1 Surface Finish will undoubtedly be influenced by the effort put into constructing the forms, and the quality of the forming material.

The specifications **require** the use of power carborundum stones or disks to remove bulges and other imperfections. Areas, which do not exhibit the required smooth, even surface of uniform texture and appearance shall be sanded with power sanders or finished using other approved abrasive means. Light sand blasting (whip blasting) is considered an approved abrasive means provided it produces the desired results (See Bridge Construction Memo 105-1.0).

To assure uniform application of this specification statewide, the Office of Structure Construction will **not** permit any sprayed-on finish or applications similar to surface painting (unless required by the contract) to be acceptable as meeting the requirements of a Class 1 Surface Finish as defined in Section 51-1.18B of the Standard Specifications.

The final surface finish of concrete barriers shall conform to the provisions of section 83-2.02D(4) of the Standard Specifications. You are directed to BCM 162-3 for more specific information on finishing concrete barriers.

c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager

METAL FORM SPREADERS

It is permissible to use metal spreaders in forming for interior box girder stems, provided the spreaders are not in contact with the girder reinforcing steel. It is not necessary that the spreaders be removed to a depth of 25 mm (1”) below the surface of the concrete.

Volume II

CONCRETE PLACEMENT DURING HOT WEATHER

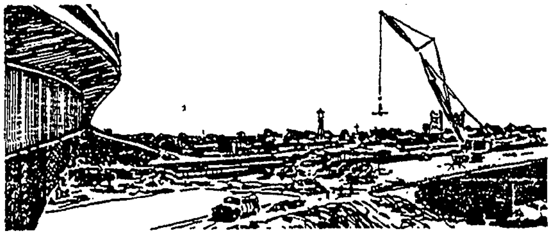
(Criteria for weather non-working days)

If concrete placement is the controlling item of work, and if all of the following applicable practices are employed and the Contractor cannot produce concrete which complies with the specified temperature, the placement of concrete shall not be commenced and a non-working day allowed as provided in Section 8-1.06 of the Standard Specifications:

1. Chipped or crushed ice is used in place of all mixing water, provided that a commercial supply of ice is available within 80 kilometers (50 miles), measured along the highway, of the project. If a commercial supply of ice is not available within 80 kilometers of the project, the coolest available water is utilized for mixing, i.e., well water instead of above ground stored water.

2. Coarse aggregate stockpiles are kept continuously wet to maximize evaporative cooling.

3. Concrete pours are scheduled so that placement of the concrete is completed prior to 12:00 noon.



BRIDGE CONSTRUCTION MEMO 105-4.0

CONCRETE - PLACE, FINISH & PROTECT

June 16, 1986

Volume II

PLASTIC CRACKING OF CONCRETE

Plastic cracks are cracks that sometimes occur in the surface of fresh concrete soon after it has been placed and while it is being finished. These cracks appear mostly on horizontal surfaces and can be substantially eliminated if preventive measures are taken to minimize the causes.

Plastic cracking is usually associated with hot-weather concreting: however, it can occur at any time when atmospheric conditions produce rapid evaporation of moisture from the concrete surface. Such cracks may appear when water evaporates from the surface faster than it can bleed (rise naturally) to the surface. This creates rapid drying shrinkage and tensile stresses in the surface that often appear as short, irregular, plastic cracks. The following conditions, singly or collectively, increase evaporation of surface moisture and increase the possibility of plastic cracking:

1. High concrete temperature
2. Low humidity
3. High winds

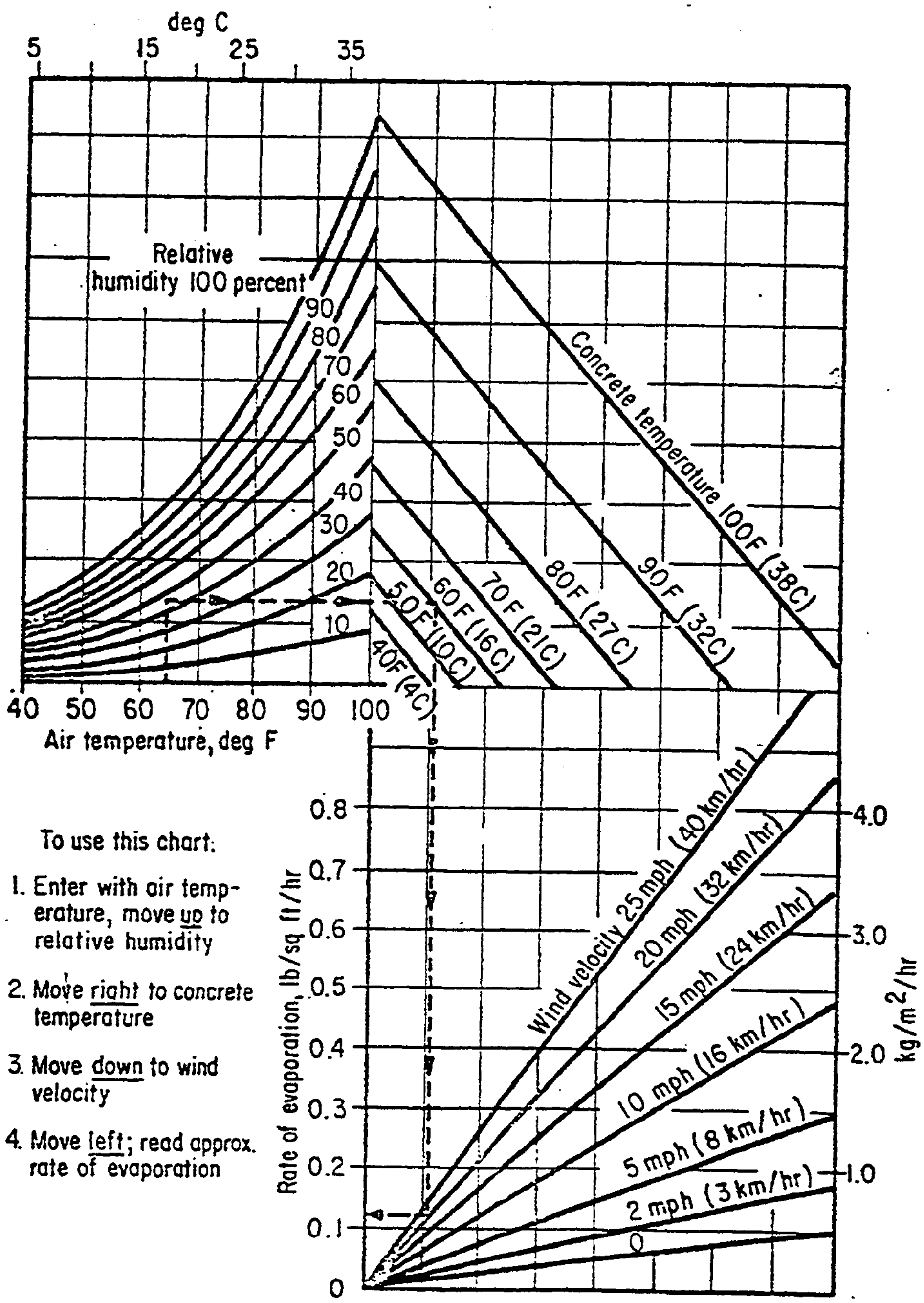
Attachment No. 1 to this Bridge Construction Memo may be used for determining when precautionary measures should be taken. There is no way to predict with certainty when plastic cracking will occur. When the rate of evaporation exceeds 0.2 lb per square foot per hour (1 $kg/m^2/hr$), precautionary measures are almost mandatory. Cracking is possible if the rate of evaporation exceeds 0.1 lb per square foot per hour (0.5 $kg/m^2/hr$).

The simple precautions listed below can minimize the possibility of plastic cracking. They should be considered while planning for hot-weather-concrete construction or while dealing with the problem after construction is started. They are in order in which they should be done during construction.

1. Moisten the subgrade and forms.
2. Moisten aggregates that are dry and absorptive.
3. Erect temporary windbreaks to reduce wind velocity over the concrete surface,
4. Erect temporary sunshades to reduce concrete surface temperatures.
5. Keep the fresh concrete temperature low by cooling the aggregates and mixing water.

6. Protect the concrete with temporary coverings, such as polyethylene sheeting, during any appreciable delay between placing and finishing.
7. Reduce time between placing and start of curing by eliminating delays during construction.
8. Protect the concrete immediately after finishing to minimize evaporation. This is most important to avoid checking and cracking. Application of moisture to the surface by fog spray is an effective means of preventing evaporation from the concrete. Fogging should be continued until a suitable curing material such as a curing compound, wet burlap, or curing paper can be applied.

If plastic cracks should appear in the fresh concrete, the cracks can be closed by striking each side of the crack with a float. However, the cracking will recur unless the causes are corrected.



Effect of concrete and sit temperatures, relative humidity, and wind velocity on the rate of evaporation of surface moisture from concrete. This chart provides 3 graphic method of estimating the loss of surface moisture for various weather conditions. To use the chart, follow the four steps outlined above, If the rate of evaporation approaches 0.2 $lb/ft^2/hr$ (1.0 $kg/m^2/hr$), precautions against plastic shrinkage cracking are necessary.

DIVISION OF STRUCTURE CONSTRUCTION *Bridge Construction Records and Procedures Manual*	B00-11
BRIDGE CONSTRUCTION BULLETIN **Approved:** **DOLORES M. VALLS, Chief** **Division of Structure Construction**	**File: BCM 105-4.1** **CONCRETE-Place, Finish & Protect** **Date: October 25, 2000** **Expires: October 25, 2001** **Supersedes: None**

Subject: Bridge Deck Construction

The purpose of this bulletin is to bring your attention to construction problems that have occurred during the construction of concrete bridge decks. Recently completed bridge decks have experienced extended concrete set times accompanied by an increase in deck cracking. Probable contributing factors are the optional use of water reducing chemical admixtures and inadequate curing procedures.

Use of Chemical Admixtures:

In general, the dosage of chemical admixtures should be used based on the cement content in a mix and **not** on the amount of cementitious material (cement + mineral admixture), unless otherwise stated in the manufacture' s published recommendations. With many chemical admixtures such as water reducers (Type A or F), the chemical reaction only effects the cement. When dosage rates of the chemical admixture (Type A or F) are based on the total amount of cementitious material, an over dose may occur, causing unanticipated results.

Water reducing admixtures (Type A or F) increase concrete set times. As an example, Grace Construction Products, WRDA 64, at a dosage rate of 3 fl. oz. per 100 lbs of cement, at 23 C, retards the set an additional 1.4 hours beyond the normal set time. A higher dosage of water reducing admixture, lower ambient or concrete temperatures, will further retard the set. In addition, the required 25% mineral admixture (flyash) typically used will retard the set an additional 1.25 hours.

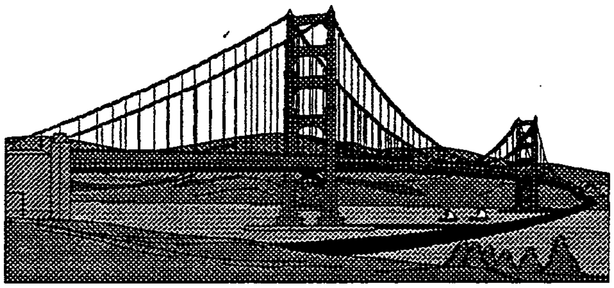
Action if set is retarded excessively:

Well proportioned concrete generally sets within 4 hours of being placed. If the concrete set is retarded excessively because of inappropriate proportioning of chemical admixtures and the contractor is unable to apply a curing medium (e.g. damp carpets or "Bur Lene" type blankets) then a continuous fine-water mist (not a spray) shall be applied to the deck. The mist shall be applied over the curing compound after it forms a membrane. This shall continue until the final medium can be applied. This should also be done for portions of the deck that are completed after normal work hours. It is advisable that the Contractor be prepared to apply the fine-water misting if application of the final curing medium is delayed by retarded set.

Below is a list of other typical causes of deck cracking. Care should be taken to assure each issue is addressed before deck concrete is placed.

	Possible Causes	**Solution**
1)	Curing compound was not mixed or applied properly.	Enforce Section 90-7.01B of the Standard Specifications which requires power driven agitators to be used.
2)	Curing compound used did not meet ASTM requirements as specified in section 90-7.01B of the 1999 Standard Specifications.	Enforce Section 90-7.01B of the Standard Specifications by taking samples of compound and sending to METS for testing.
3)	Rapid evaporation of surface moisture occurs due to high temperatures, windy conditions, and or low humidity.	Apply a fine-water mist as specified in section 90-7.01B of the 1999 Standard. (See BCM 105-4.0 for info on evaporation rates)

c: BCR&P Manual Holders
EDavisson, ESC Deputy Director
RPieplow, Construction Program Manager
TRut, Chief, Division of Structure Maintenance & Investigation
RLand, Chief, Division of Structure Design (Acting)
PStolarski, Chief, Division of Materials Engineering and Testing Services



Volume 2

BRIDGE CONSTRUCTION MEMO 105-5.0

CONCRETE-PLACE,FINISH & PROTECT

December 2, 1996

USE OF “BURLENE” FOR WATER CURE

The use of "Burlene” (natural or synthetic fiber) is acceptable for water curing structure concrete. Included with this memorandum is a brochure from Burlene for your information.

Conditions for its use are as follows:

1. There are no restrictions on which structure elements can be water cured with this material This includes concrete columns and retaining walls.

2. The burlap side of the material is to be placed next to the concrete. On decks, the burlene shall be secured by weighing down or other methods which ensure a proper seal and protection against wind. On columns and retaining walls the burlene shall be secured at the top, bottom, discontinuous edges, and loosely secured at mid-height or at no more than twenty feet on center for columns over forty feet in height. In addition, on flared or unusually shaped columns or walls the material shall be secured in such a fashion that the burlene is within three inches of the surface of the concrete at all points along the surface being cured. Joints shall be folded and secured by ties, staples or stitching as necessary to ensure a tight seal when curing columns and walls. Weighted lap splices are acceptable on decks and other similar surfaces.

3. Cure water is to be applied under the burlene (between the plastic sheeting and the concrete). On columns and walls, a soaker hose, or comparable device, which will provide an even water distribution completely around the perimeter of the surface being cured shall be permanently installed for the entire length of the cure period Water shall be applied as necessary to keep the burlene and concrete surface moist at all times.

4. The temperature of the concrete should be monitored during hot weather applications of “Burlene” curing. Field observations indicate “Burlene” type materials do not provide the cooling that evaporation of water from a continuously wetted retaining medium (i.e. rug or mats) provide. Concrete surface temperatures of 140° F or greater must be prevented.

The burlap side of the material should be moist at all times. On decks, you should anticipate a need for more frequent cure water applications over that required by the use of traditional rugging. On columns and retaining walls a continuous application of cure water may be required

THE 'PROS' KNOW AND USE . .

THE MODERN *Wet Cure* METHOD

with *Bur Lene*® concrete curing blankets!

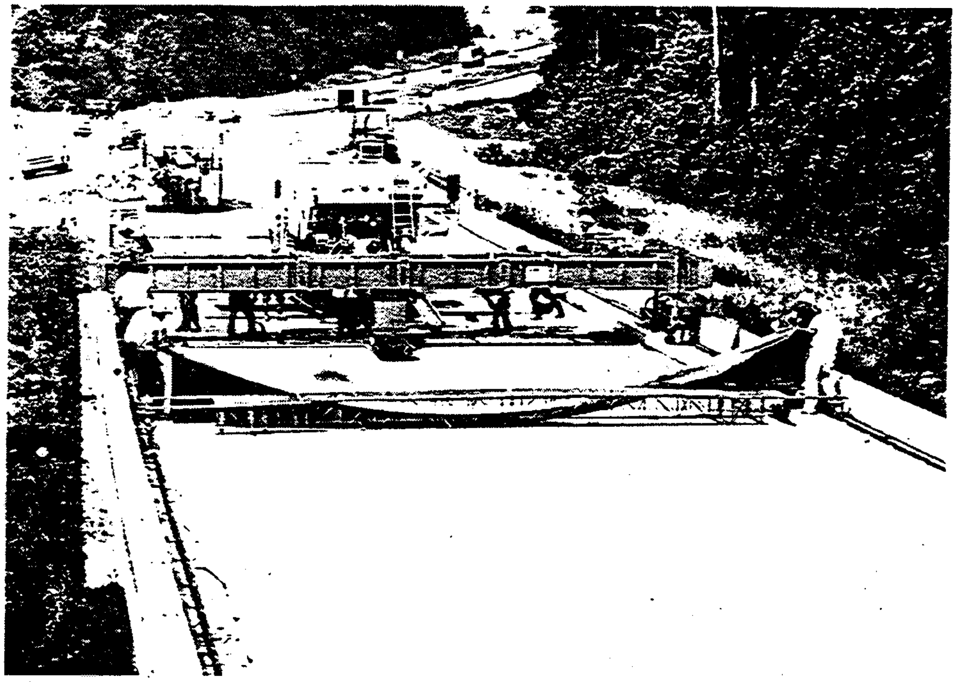
Bur Lene PASSES SPECIFICATIONS IN MOST STATES AND FEDERAL AGENCIES

Number One for:

- SLABS
- RAMPS
- DECKS
- PIERS

The Best for:

- COLUMNS
- FOOTINGS
- WALKS
- FLOORS



INSTRUCTIONS:

Lay out *Bur Lene* Blankets (burlap side down), lapping 6" on top of slab, and allowing sufficient lap over edge, to seal-in.

Fogspraying concrete and/or pre-wetting burlap side of *Bur Lene* may be helpful in most applications.

FOR LONGER LIFE

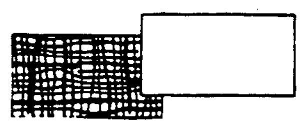
After each use, expose burlap side of blanket to sun or air, allowing burlap to dry, then continue the application over and over—following this procedure.

THE 'PROS' KNOW THE ANSWER . . .

Bur Lene® concrete curing blankets save 4 ways:

- ✓Save Time
- ✓Save Water
- ✓Save Labor AND...
- ✓Saves on Costly Replacement.

Here's The Secret of *Bur Lene* Effectiveness!



Bur Lene has the strength and water-distribution features of 10 oz. burlap combined with the moisture-retention and reflective features of 4 mil white opaque poly, extruded into burlap.

Bur Lene is the revolutionary new method of curing concrete. Designed specifically for the curing of concrete slabs, such as bridge decks, highways, buildings, airport runways, military installations, etc., *Bur Lene* offers two important functions that make the big difference in results.

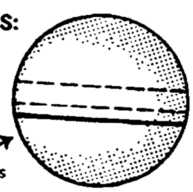
First—*Bur Lene* burlap layer distributes water evenly (much like the wick in an oil lamp). Second—*Bur Lene* white opaque poly layer helps hold in the moisture and reflects the sun's rays to help keep the concrete wet and cooler at a more even temperature.

Bur Lene has already proved its great advantages in many areas, and is being approved as one of the most effective, most economical methods of concrete curing on the market today.

AVAILABLE STOCK ROLLS & BLANKET WIDTHS:

3 1/3'–5'–6 2/3'–10'–15'–20'
Less seaming allowance 2" per seam.
Cut to any length required

STOCK ROLLS – 100' Long



SEAMS: Flat Seams Double Sewn to keep blankets laying snug and flat on concrete surface.

MANUFACTURED BY

MAX KATZ BAG CO., INC.

CONSTRUCTION PRODUCTS DIVISION

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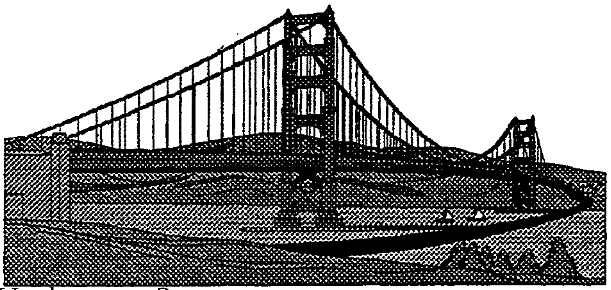
Bridge Construction Memo 105-5.0
Attachment 1 (12-02-96)
Sheet 1 of 1

2/94 5M

Originators Bur Lene Concrete Curing Blankets



USE OF "VISQUEEN" FOR COLUMN CURE

"Visqueen" (plastic sheeting) use will be allowed for column curing under the following conditions:

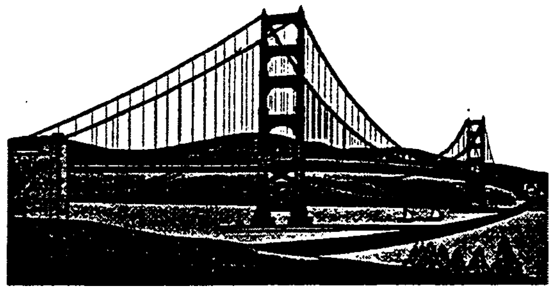
1) Visqueen shall be new or in near new condition, without tears or holes. The thickness shall be 10 mils, minimum, and shall be achieved in one layer of material.

2) Visqueen sheets are to be adequately secured at the top, bottom, discontinuous edges, and at mid-height, or at no more than twenty feet on center for columns over forty feet in height.

3) Joints in visqueen shall be folded and secured by tape, ties, clamps, or stitching as necessary to ensure a moisture-proof seal. The sheathing shall be fastened such that it will remain within 3 inches of the concrete surface at all times.

4) Visqueen is to be used in conjunction with the application of water. Cure water is to be applied between the visqueen and the column by means of a soaker hose or comparable device which will provide an even water distribution completely around the perimeter of the column and shall be installed for the entire length of the cure period. Cure water should be applied at least twice daily and as required to keep the column surface moist, at all times.

In regions of extreme high temperatures, if the surface temperature of the column under the visqueen cannot be maintained below 140° F, the use of visqueen shall be discontinued and one of the other curing methods allowed under Section 90-7.03 of the Standard Specifications shall be used. Refer to BCM 105-5.0 for permissive use of burlene for water cure of other structure elements.



BRIDGE CONSTRUCTION MEMO 110-0.0

COMPUTER INFORMATION

April 2, 1997

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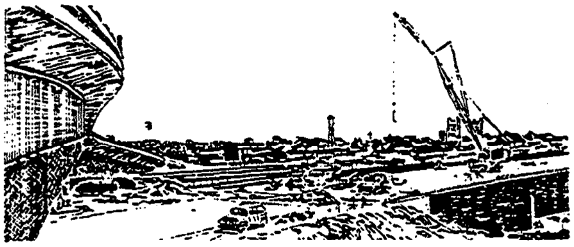
110-4.0	04-02-97	UNAUTHORIZED MODIFICATIONS TO OSC FIELD COMPUTERS

RALPH P. SO-A, Chief
Office of Structure Construction



Volume II

ELECTRONIC COMPUTER PROGRAMS

General

Electronic computer programs have been developed to facilitate the design and layout of bridge structures. Some of these programs are applicable to field construction problems.

Attached is a list of computer programs (Attachment #l) which are in general use by the Office of Structure Construction. The list includes the Bridge Computer Manual index number, the date that the computer program was issued, and the title of the computer program. All of the listed computer programs are included in the Bridge Computer Manual which contains a complete description of the program and instructions for its use.

On the attached list, those computer programs which have most frequent application to field work are identified by an asterisk (*).

Field construction personnel who want to use any of the listed computer programs may obtain a description of the program and instructions for its use by submitting a written request to the Office of Structure Construction, Attention: Lloyd Johnson.

EDP By Wire Transmission

All routine highway and bridge engineering computer services available by mail to and from Sacramento are also available by wire transmission from and to each District Office. Structure Construction personnel whose locations permit ready access to a District Office are invited to take advantage of this service.

Check with the District Data Processing Coordinator to make sure that he/she is familiar with the Job Control Language (JCL) and that a method exists to enter the program in the system. If not, mail the filled in computer input forms to the Office of Structure Construction in Sacramento, Attention: Lloyd Johnson.

29F

Computer Service

Any questions concerning computer service should be referred to the Office of Structures Design, Bridge Computer Services Section in Sacramento, phone ATSS 454-9235 or (916) 323-9235.

Field personnel should check to see if a computer program on hand is the current edition before using it.

Filled in computer input forms for bridge programs to be processed in Sacramento should be sent to:

Office of Structure Construction
Attn: Lloyd Johnson
P.O. Box 942874
Sacramento, CA 94274-0001

Lloyd will assign District, Group, Batch, and Problem Number, and will arrange for the programs' run and its return to field personnel.

ELECTRONIC COMPUTER PROGRAMS IN GENERAL USE

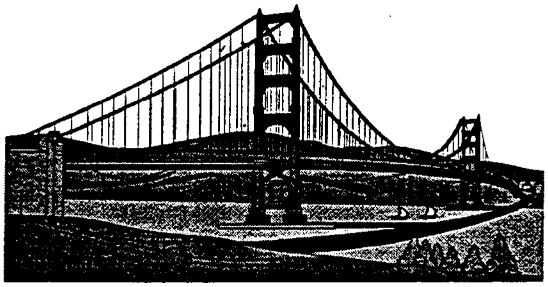
BRIDGE COMPUTER MANUAL

Index Number	Date	Program
* 2-1	6-79	Traverse and Horizontal Alignment
* 2-2	1-72	Vertical Alignment
2-2.1	l-72	Vertical Alignment Plot
*-2-3	4-85	Layout Plot
* 2-4	3-72	Bridge Deck Geometrics (See Note 1)
* 3-1	2-75	Frame System (See Note 2)
3-1.5	1l-73	Frame System Update
3-3	1-72	Bent Analysis
3-3.1	l-72	Sent Analysis Supplement #1
3-4	3-72	Tunnel Arch Analysis
4-1	2-73	Section Properties by Coordinates
4-2	3-72	Girder Deflections
5-1	6-81	Column Design by Ultimate Strength
5-2	8-72	General Column Analysis
6-1	1l-80	Prestressed Girder Analysis
6-2	8-72	Continuous Prestressed Girder Analysis
7-1	12-72	Composite Girder Design
7-2	3-72	Composite Girder Analysis
7-3	2-73	Composite Girder X-Section, Analysis
* 9-1	1-72	Six-factor Computations
9-2	5-80	Superstructure Concrete quantities
* 9-3	1-72	Reinforcing Steel

Note 1 - This program may be used to obtain 4-scale contour plots, Refer to Bridge Construction Memo 2-4.0 for additional information.

Note 2 - This program may be used to determine falsework girder deflections and reactions.

*Computer program which has most frequent application to field work.



BRIDGE CONSTRUCTION MEMO 110-2.0

COMPUTER INFORMATION

April 2, 1997

Sheet 1 of 1

Volume II

REQUIRED USE OF COMPUTER SECURITY CABLES

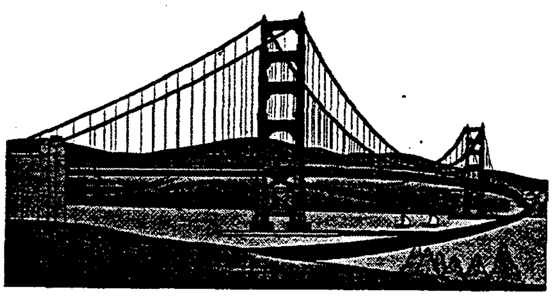
It **is Office of Structure Construction policy that the CPU, monitor, and printer be** anchored with **a plate and cable system to the desk or table. Any less of a procedure places the computer system at risk.**

The recipient of the computer system will be held accountable for protecting the computer equipment assigned to them and this **requires (as a minimum) the anchoring of the computer system components to the desk or table top as described above. It is prudent that this anchoring be completed the same day the computer is received or is moved into a new office.**

Please verify that OSC computers in your office have been properly secured in conformance with this memo. If security parts are missing, (moving, replaced components etc.) please contact one of the persons listed below for replacement parts.

The security cable system consists of four adhesive plates, a 6 foot cable, special washer and a padlock. An adhesive anchor plate must be installed on each computer component (CPU, monitor, printer) and the back or bottom of the desk. The computer system should be so arranged that a single cable connects all four plates together.

Replacement parts may be obtained by contacting either the OSC system specialist at **(916) 227-8401 or the OSC computer operations engineer at (916) 227-8980. Please provide your name, the Caltrans No. (gold CT No. tag) of the CPU and what parts are missing. Order only for your current needs.**



BRIDGE CONSTRUCTION MEMO 110-3.0

COMPUTER INFORMATION

April 2, 1997

Volume II

COMPUTER HARDWARE UPGRADE POLICY

Computer hardware upgrades are not permitted, with minor exceptions, to OSC issued computers. Authorized upgrades must have written approval in advance of the work being performed from the OSC Computer Operations Engineer, (916) 227-8980.

BRIDGE CONSTRUCTION MEMO 110-4.0

COMPUTER INFORMATION

April 2, 1997

Volume II

UNAUTHORIZED MODIFICATIONS TO OSC FIELD COMPUTERS

Routine system checks have uncovered that a number of unauthorized modifications are being made to individual PCs assigned to OSC field off ices. These have included hardware modifications, reconfigured system commands or files, over-written set up files that make the system inoperative, and assorted "tinkering" that make the supplied system nearly unrecognizable. In some cases, new personnel have assumed excessive freedom and customized the PCs.

Also discovered have been illegal installation of non-licensed software on the OSC system, and illegal transfer of OSC system software to other PCs in the field office.

These unauthorized/illegal modifications have compromised our system's ability to do the work it was designed to do. The benefits of standardization to multi-users are lost, and computer support is made difficult. Other PCs have required complete reformatting of the hard drive and re-installing the entire software system.

OSC policy for PC use is as follows:

- Use the system as supplied.
- Properly install the updates issued to correct errors and modify programs.
- Do not modify the system configuration.
- Do not install illegal software, or software from other computer systems.
- Do not install copyright software from the OSC system to outside systems.
- Request help and assistance for special situations directly from the System Specialist at (916) 227-8401.

Department policy on illegal use of software is per attached memo dated May 12, 1989 and signed by Deputy Director Carolyn Peirce Ewing.

Individuals who cannot or will not follow policy and basic common sense will have their OSC PC system reassigned to someone else and face potential disciplinary action.

Attachment

M e m o r a n d u m

To: All Employees

Date: May 12, 1989

File:

From: DEPARTMENT OF TRANSPORTATION
DIRECTOR'S OFFICE

Subject: Illegal Use of Microcomputer Software

Software sold for use on microcomputers is copyright-protected in most cases. When it is purchased by a Caltrans employee using Caltrans funds, we honor that protection by not making copies of the product. Software providers are aggressively filing lawsuits against those who duplicate protected software.

In a recent court case filed against the California State Colleges and Universities System, a software company claimed that their copyright protected software was being copied on State microcomputers without separate purchase and license. The State contended that a State agency cannot be prosecuted using Federal Law as the basis of the suit, and the State won the case. On appeal, higher courts found again for the State's position. What this means is that the software vendor is now pursuing the individual professors <u>and</u> their managers for personal liability.

The California State Administrative Manual cites the following:

> "In the conduct of their operations and in the accomplishment of the policies stated above, State agencies and their employees shall employ information technology in a legal and ethical manner consistent with government statutes, rules and regulations. Information technology shall not be used for purposes that are unrelated to the agency's mission or that violate State or Federal law. Contract provisions, including software licensing agreements, shall be strictly followed."
> (S.A.M. Section 4820 - ETHICS)

> "Management responsibility for the use of each personal computer, as well as for the security of data, hardware and software, resides with the individual managers who are otherwise responsible for the personnel who regularly use the computer."
> (S.A.M. Section 4990.1 - Management Responsibility)

"Software license agreements must be strictly adhered to. Proprietary software cannot be duplicated, modified or used on more than one machine, except as expressly provided for in the manufacturer's license agreement."
(S,A.M. Section 4990.1 - Software License Agreements)

Caltrans has added the phrase "... or other modified agreements negotiated with. the software manufacturer." ... to this last policy.

The above information was provided to Caltrans Executive Management in a February 17, 1989 letter from Marlin Beckwith, Investigations pursued by some of those managers indicated that in some instances software may have been copied.

All managers, supervisors and employees should investigate their own work environment for potential copyright abuses and assure complete compliance with State regulations. After July 15, 1989, violations of microcomputer software copyrights or State regulations will result in disciplinary action being taken by the Department.

CAROLYN PEIRCE EWING
Deputy Director
Administration and Transportation
Programs

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 110-5 COMPUTER INFORMATION**
Approved: Original Signed by Dolores Valls **DOLORES M. VALLS, Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: N/A** **Supersedes: None**

Subject: Computer Repair Procedure

The Offices of Structure Construction's (OSC) policy for obtaining computer repair service is to first call one of the OSC PC System Specialists listed on the OSC Intranet at http://oscnet.dot.ca.gov/oscnet/ under "SC People", then "HQ People". The PC System Specialist will work with the user to correct the problem, determine if a hardware service call is required, and contact the appropriate repair organization. The computer user is not to directly contact the repair provider unless directed to do so by the PC System Specialist.

The PC System Specialist will request specific information from the user, such as:

CT Number ______________________
Serial Number ______________________
Make ______________________
Model ______________________
OMRS Number (if applicable) ______________________
Contact Person ______________________
Field Office Address ______________________
Field Office Telephone ______________________

If the component is under warranty, the manufacturer's authorized vendor will be contacted. Otherwise, the PC System Specialist will contact the Department of General Services, Office of Machine Repair Services (OMRS).

For service by OMRS, it is essential that a contact person be at the field office during the repair time to review and sign the Office Machine Service Order. Check that the following are included: CT number, serial number, billing cost code 60048, the service technician's total hours, the hourly rate, and the cost of the parts. Part costs are on the packaging.

Should the service technician inquire if upgrades should be installed while they are working on your machine, the answer is "**NO**". These seemingly simple upgrades are chargeable to OSC's equipment budget and are ***not*** to be performed without prior written authorization from the OSC PC System Specialist.

Send a copy of the Office Machine Service Order to the OSC Headquarters at 1801 30th Street, Sacramento, CA 95816, Attention: Information Technology Branch Chief, for input into the equipment maintenance database.

c: BCR&P Manual Holders
BGauger, HQConstruction Program Managers

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 110-6 COMPUTER INFORMATION**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: None** **Supersedes: BCM 110-6 Dated October 1, 1998**

Subject: Use of Internet and Electronic Mail (e-mail)

The Offices of Structure Construction policy regarding the use of the Internet and Electronic Mail (e-mail) is no different from the policy set forth in Deputy Directive Number 54R, "Information Technology Use Standards", dated April 30, 2001. A copy of this policy is available on the Caltrans Intranet at:
http://adsc.caltrans.ca.gov/bfams/deputydirectives/

Employee's Responsibility

Offices of Structure Construction (OSC) personnel shall read, understand and adhere to the policy set forth in Deputy Directive Number 54R.

Supervisor's Responsibility

It will be the responsibility of Supervisors within the OSC to:

1. Provide OSC employees with a copy of Deputy Directive Number 54R.

2. Ensure that their employees have read, understand and adhere to the policy set forth in Deputy Directive Number 54R.

c: BCR&P Manual Holders
Consultant Firms
R. Pieplow, HQ Const.

Volume II

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112-5.0	10/31/2003	METHACRYLATE DECK CRACK TREATMENT

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

BRIDGE CONSTRUCTION MEMO 112-1.0

DECKS

Septenber 2, 1974

Volume II

LONGITUDINAL DECK CONSTRUCTION JOINTS

EXPANDED METAL LATE FORMS

Expanded metal lath may be used to form longitudinal deck construction joints provided the joint is constructed in compliance with the following requirements.

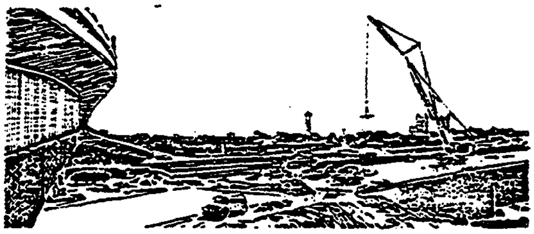
1. A continuous wood strip, the height of the bottom steel clearance, shall be securely fastened to the deck form.

2. Another continuous wood strip, the height of the top steel clearance, shall be fastened securely on top of the top steel, This strip shall be set to grade and held firmly in place by means independent of the reinforcing steel.

3. The expanded metal lath shall be securely fastened between these wood strips.

4. Concrete along the face of the metal lath form must be cured by the water method.

5. Before the second pour is made, wood strips must be removed, and any fractured concrete removed. Metal lath that is firmly embedded in concrete may remain in place provided it has the same clearance from the concrete surface as the specified steel clearance. Any loose mesh must be removed.



Volume XI

TESTING BRIDGE DECK SURFACES FOR COMPLIANCE WITH THE STRAIGHTEDGE OR PROFILOGRAPH REQUIREMENTS

General Information

All deck surfaces must be tested to be assured of compliance with the Bridge Deck Finishing Specifications. Note that the Specifications have the following requirements:

(1) Concrete decks or concrete approach slabs, which are to be covered with one inch or more of another material, will be tested both longitudinally and transversely with a 12-foot long straightedge.

(2) The completed roadway surfaces of structures, approach slabs, and the adjacent 50 feet of approach pavement will be tested with a bridge profilograph in the longitudinal direction, and with a 120-foot straightedge in the transverse direction.

Note that if a concrete deck were specified to have a two inch AC overlay, it would be necessary to test the concrete deck both longitudinally and transversely with a 120-foot straightedge prior to placing the AC overlay; and then after placing the overlay, the AC surface should be tested longitudinally with a bridge profilograph and transversely with a 120-foot straightedge.

Straightedging Deck Surfaces

When the Specifications require the use of a straightedge to check deck surfaces, the decks must be systematically checked with a 120-foot straightedge over the entire area. This should be done as soon as the concrete can be walked upon without damaging the deck surface, and should be completed prior to the time that the deck surface is covered with rugs, mats, or other material that would interfere with the straightedging operations. Any places which do not meet specifications should be marked with red spray paint.

It is the Contractor's responsibility to straightedge the deck while the concrete is wet. Structure Construction personnel should not straightedge wet concrete except under unusual circumstances.

Profilograph Testing of Deck Surfaces

When the Specifications require the use of the Bridge Profilograph to check deck surfaces, the deck will be tested with the profilograph in accordance with the Test Method No. Calif. 547. High points in excess of 0.25 inch should be marked with red spray paint. It shall be the Contractor's responsibility to schedule the profilograph testing operations. The Contractor shall request testing at least 7 days prior to need, and shall ensure that the entire area to be tested has been cleared and cleaned of all obstructions.

Because the Districts currently have the expertise in maintaining highway profilographs, they will also have the responsibility for storing and maintaining the bridge profilographs. Therefore, when bridge profilographs are needed, they should be obtained from the District Construction or Materials Department. The method of handling the profilographs varies somewhat in the various Districts. It will therefore be necessary for the Structure Representative to become familiar with the procedures, and persons to contact, in the District in which he is working. When not actually in use on a project, the profilograph should be returned to the proper District authority.

Letter to Contractor

As soon as possible after testing the deck with the profilograph and/or the straightedge, a letter should be written to the Contractor advising him that the deck has been checked for compliance with the profilograph requirements and/or the straightedge requirements. The letter should describe the specific locations that fail to meet the straightedge specification, or describe any deficiencies in meeting the profilograph specification. The letter should state that the specific deficiencies must be corrected before the contract can be accepted. After the deficiencies have been corrected, or if the entire deck initially complies with the applicable straightedge or profilograph requirments, then write the Contractor a letter stating that the deck was checked and that it complies with the profilograph requirements and/or the straight edge requirements whichever is applicable. (See Attachment No. 1 for a sample letter relative to concrete decks or concrete approach slabs which are to be covered with one inch or more of another material. See attachment No. 2 for a sample letter relative to the completed surfaces of bridge decks, approach slabs, and adjoining 50 feet of approach pavement.

The Specifications allow the Engineer to point out a contract deficiency to the Contractor at any time. However, once the Engineer has completely satisfied himself that the deck surface complies with the Specifications, and has given the Contractor a letter advising him of this, it is the mark of an ethical Engineer to consider the matter closed.

(Sample of letter to be sent to the Contractor for concrete decks or concrete approach slabs which are to be covered with one inch or more of another material.)

The finished surface of the deck concrete at ____________ ____ Bridge No. ____________ has been tested for compliance with the straightedge requirements in Section 51-1.17, "Finishing Bridge Decks," of the Standard Specifications.

(USE EITHER)

All areas tested were found to comply with the specified straightedge requirements.

(OR)

Areas that do not meet the straightedge requirements have been marked, and ate located as noted below:

(EXAMPLES) Sta. 300+52 (5 ft. to 15 ft. from Rt. EOD)

Sta. 301+60 (10 ft. from Rt. EOD)

Hinge in Span 3 (Entire bridge width)

Transversely across longitudinal construction joint. (Sta. 300+10 to 302+10)

These deficiencies must be corrected before the ________ __________________________ overlay is placed.
(describe overlay)

Notification shall be given to the Resident Engineer prior to performing the corrective action.

(Sample of letter to be sent to the Contractor for completed surfaces of bridge decks, approach slabs, and adjoining SO feet of approach pavement.)

The completed surface of (bridge deck) (approach slab) (adjoining 50 feet of approach pavement) at Bridge No. ________ Bridge Name ____________________ has been tested for compliance with the profilograph requirements and the transverse straightedge requirements of the Standard Specifications.

(USE EITHER)

All areas tested were found to comply with the specified profilograph (and) (or) the transverse straightedge requirements.

(AND/OR)

The profilograph trace indicates that there are high points in excess of 0.25 inch and that the profile count exceeds 5 per hundred feet. High points in excess of 0.25 inch have been marked with red spray paint. A profile trace is available for your examination at the Resident Engineer's office. The completed surface must be ground in accordance with the requirements in Standard Specification Section 42, "Groove and Grind Pavement," until the specified smoothness tolerances are met.

(AND/OR)

Straightedging in a transverse direction indicated that the roadway surface varied more than 0.02 foot from the lower edge of a 120-foot long straightedge at the following locations: Areas that do not meet the straightedge requirement have been marked, and are located as noted below:

(EXAMPLE) 4 foot from the left EOD between Sta 300+00 and Sta. 300+75.

Longit. coast. jt. at center of left bridge between St3. 300+50 and Sta. 301+10.

These deficiencies must be corrected before the contract can be accepted. Notification shall be given to the Resident Engineer prior to performing the corrective action.

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 112-2.1 DECKS**
Approved: ______________________ **R. P. SOMMARIVA, Chief Division of Structure Construction**	**Date: September 15, 1999 Expires: None Supersedes: None**

Subject: PROFILE COUNTS OF DECK PROFILOGRAPHS

This Bulletin clarifies references to 'profile count' in Section 51-1.17, "Finishing Bridge Decks", of the Standard Specifications. The **1992 edition** of the specification states that:

> The surfaces shall have a profile trace showing no high points in excess of 0.25-inch and the portions of the surfaces within the traveled way shall have a profile count of **5 or less in any 100-foot section**.

This 'profile count' refers to the summation of the height of scallops that are greater than 0.03" high and measured to the nearest 0.05" and expressed in tenths of inches. Thus, a count of 5 would reflect 0.5" of scallops recorded in a 100 ft section of profile trace. This is outlined in California Test 547.

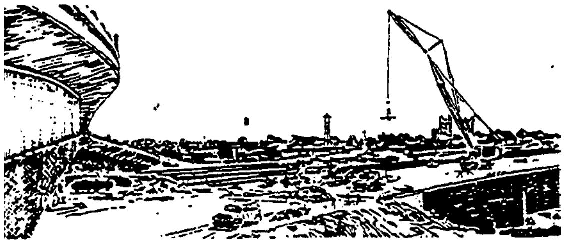
The **1995/1999** editions of the specification states:

> The surfaces shall have a profile trace showing no high points in excess of 6.35 mm, and the portions of the surfaces within the traveled way shall have a profile count of **13 or less in any 30-m section**.

This profile count refers to the same criteria as listed above for the 1992 specification with the exception that the value is expressed in millimeters. Thus, if you had a count of five after following the procedure in California Test 547, you would multiply this value by 2.54 to obtain the 'metric' value of the profile count.

Use the templates required by California Test 547 and perform the evaluation in accordance with the test method. When reporting the 'count', you **multipy the count** (which represents tenths of an inch in a hundred feet) **by 2.54** to obtain the 'count' to compare to the metric specification reference (i.e. a count of 7 using the templates would equate to a metric profile count of 17.8).

c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager



SUPPORTS FOR FINISHING MACHINES

RAILS

The Standard Specifications require that "Rails for support and operation of finishing machines or hand-operated strike-off devices shall be completely in place and firmly secured for the scheduled length for concrete placement before placing of concrete will be permitted."

"Rails" as used in this specification shall be interpreted to be the continuous structural members which are readily adjustable for elevation, and which support the finishing equipment. These "Rails" must be in place for the entire length of the pour.

Various devices may be placed on top of these rails for purposes of wearing or spacing, Such devices do not have to be full-length and may be "leap-frogged" around the finishing equipment.

TESTING BRIDGE DECK SURFACES FOR COMPLIANCE WITH COEFFICIENT OF FRICTION REQUIREMENTS

Testing of bridge deck surfaces shall be done in accordance with California Test Method 342. A minimum of one Bridge Deck Skid Resistance Test should be performed on each bridge deck. The test shall be performed at a location which is representative of that portion of the deck surface exhibiting the poorest coefficient of friction. Once the smoothest area has been tested more tests will not be required unless in the opinion of the Engineer the test results are not representative or if several areas remain in question. Testing of the skid resistance of a small deck in a remote area may be waived provided the Bridge Representative makes a visual inspection of the deck surface, and determines that the deck surface may reasonably be expected to have a coefficient of friction of 0.35 or greater according to the Standard Specifications.

Refer to Section 51-1.17 of the Standard Specifications for information relative to time of testing, required coefficient of friction, and corrective measures required.

To arrange for testing contact the Pavement Evaluation and Testing Branch of the Division of New Technology, Materials, and Research at (916) 227-7299 or ATSS 498-7299. You are requested to provide 14 days prior notice.

BRIDGE CONSTRUCTION MEMO 112-5.0

DECKS

October 31, 2003

Volume II

Methacrylate Deck Crack Treatment

Methacrylate resin is commonly used to seal cracks in concrete bridge decks. Methacrylate treatment is designed to be a crack sealer only. It is not a surface overlay. Typically, the resin is applied to the deck surface by hand or mechanically sprayed and spread with a broom or squeegee. Sand is applied shortly after resin placement to provide short-term skid resistance.

Current specifications now require methacrylate treatment in lieu of epoxy injection to repair excessive shrinkage cracking in newly constructed bridge decks. Treatment is required when the measured crack intensity is greater than 5 linear meters (16 ft.) of deck cracking exceeding 0.5 mm (.02 in.) in width in any 50 m^2 (500 $ft.^2$) area of deck. The treatment shall extend transversely, the entire width of the new deck (barrier rail to barrier rail) and longitudinally, 1.5 meters (5 feet) beyond the furthest single crack that exceeds 0.5 mm (.02 in.), outside the 50 m^2 (500 $ft.^2$) portion.

Deck crack intensity measurements are to be performed after concrete cure and prior to both post-tensioning and/or falsework release. Deck crack treatment shall take place after the grinding operation.

In general, the application of methacrylate on a bridge deck is a simple process. However, careful inspection is needed to assure the treatment is effective and that the treated surface maintains a desirable roadway surface condition.

Attachment No. 1 contains inspection guidelines to assist OSC personnel when inspecting a methacrylate application.

Methacrylate Deck Treatment Inspection Guidelines

<u>Prior to Starting Work:</u>

- Review and approve the contractor's program for public safety associated with the use of methacrylate resin.
- Forward Form CEM 3101 (Notice of Materials to be Used) to METS.
- Verify the methacrylate was tested and released by METS (TL 101)
- Verify the sand meets the specification requirements.
- Contact the Skid Test Unit and make them aware of your tentative skid testing requirements. Skid test shall be performed on the test section, outside of the traveled way, prior to starting production work. The pre-production skid test will demonstrate the contractor's ability to obtain a desirable deck surface within approximately the same time frame and conditions as the production work. It will also provide a visual reference for the Caltrans inspector(s) when evaluating the roadway surface, after methacrylate and sand placement, for sufficient frictional resistance. Skid test on the production work (i.e., within the traveled way) should be performed as necessary to assure that the roadway has sufficient frictional resistance. Depending on the experience level of the inspector, this may entail performing a skid test after every methacrylate placing operation. At a minimum, one skid test per bridge should be recorded in the project documents. Additional skid test may be necessary if conditions change or deficiencies are observed during the work. For skid test contacts see: http://onramp.dot.ca.gov/hq/esc/METS/opr/skid_testing/index.htm
- Hold a meeting with the contractor to discuss, at a minimum, required test section, skid testing, application equipment, safety, abrasive cleaning methods, and a contingency plan if the resin does not cure in time.

<u>During Construction Operations:</u>

- Prior to treating deck areas within the traveled way, the contractor is required to treat a test area. Treatment of the test area must be done under the same conditions with the same materials and equipment. The test area must pass all of the acceptance criteria (i.e., tack free, sand adhesion, dry tile test, and coefficient of friction) prior to allowing the contractor to start treating the deck area within the traveled way. The main purpose of conducting a test area is to obtain the necessary information (i.e., application rate, initiator/promoter amount, set time, coefficient of friction etc.) to assure that the work within the traveled way can be completed without disruption to the traveling public.

- The concrete deck surface must be cleaned prior to methacrylate application. Sand blasting or shot blasting is typically used to prepare the deck surface. Check the contract's special provisions for the specified deck cleaning method.

- Prior to applying the methacrylate, weather conditions and deck surface temperature should be checked. Current specifications limit the relative humidity to 90 percent or lower at the time of treatment. The deck surface needs to be dry and below 38 degrees centigrade (100^{o}F).

- A tile test may be performed for additional verification that the methacrylate resin is tack free prior to permitting traffic onto the treated area. The tile test involves placing the resin on a .3-meter (12 in.) square smoothed glazed tiles for each batch of resin mixed. The test tile is placed next to the treated area and monitored through the curing process. Sand is not applied to the test tiles.[1]

---

[1] As of the issue date of this BCM, the current, "Deck Crack Treatment" Standard Special Provision is in revision. The new version will require the contractor to perform a tile test.

- Methacrylate resin can be applied by hand or with mechanical equipment. If mechanical sprayers are used they must be the "airless" type. Compressed air spray application causes too much mist and cannot be used. Typical application involves hand placing with buckets and spreading with a squeegee or broom.

- The resin must be placed within 5 minutes of the initiator being mixed. Excess material needs to be distributed (by a squeegee or broom) within 10 minutes of application. The redistribution of the excess material is important to avoid thick glassy spots that may reduce the coefficient of friction. Spread rates are determined by the engineer and will vary depending on the surface condition and aggregate type. As a rule of thumb, 2.45 m^2/liter (100 ft^2/gal) is a good starting point for normal concrete. Lightweight concrete takes less (about 1.6 m^2/liter (65 ft^2/gal)), and very dense concrete can take up to 4.29 m^2/liter (175 ft^2/gal). As a "rule of thumb" the surface should be slightly wet 20 minutes after application. If dry after 20 minutes, increase the resin. If ponding is evident, reduce the amount of resin. It is essential that the resin remains fluid long enough (40 to 90 minutes) for the cracks to be filled. If rapid gelling occurs the material should be rejected.

- Methacrylate resin shall only be applied to the deck area. The contractor is required to protect or avoid placing resin on other parts of the structure (e.g., barrier rails, joints, drainage facilities, etc).

- No sooner than 20 minutes after the resin is placed, sand shall be applied to the treated area. Sand is applied to temporarily increase skid resistance. Careful inspection of the deck surface after the sand application is needed to assure that the sand adheres to the deck. Any areas found absent of sand adhesion shall be abrasively blasted. Vacuum attachments must be used during abrasive blasting operations.

Prior to opening the treated area to traffic, the following requirements must be met:

1) The treated surface is tack free (non-oily).
2) The applied sand resist brushing by hand. If areas exist where the sand does not adhere and appears "shiny" or "glassy", the contractor must abrasively blast to roughen the surface. Excess sand must be removed.
3) Roadway surface is non-slick and exhibits a texture with a minimum coefficient of friction of 0.35.
4) The test tile(s) from the tile test is tack free (if applicable).

Typical problems associated with methacrylate deck treatment operations

The tack, or the oiliness, of methacrylate resin can create serious problems, especially in cold night closures. Opening traffic lanes prior to the complete cure of the resin can cause the oiling of cars and/or traction related traffic accidents. This issue is due to "oxygen inhibition" of the top surface. Methacrylate resin cures from the lack of oxygen, thus the exposed surface tends to cure last. Even if the bulk of the resin sets up and can resist penetration with a screwdriver, the surface can be still covered with an oily sheen. Modern methacrylate resins contain additives to prevent this phenomenon.

Heat and sunlight can cause methacrylate to set faster. Occasionally, the resin will set before the material is spread. This causes the worst case for crack sealing as it prevents the resin from properly flowing into the cracks. Additionally, if the resin sets prior to applying the sand, glassy spots will exist. The glassy areas may have reduced skid resistance and remedial work to repair these areas would be required. Remedial methods that have been proven effective are to abrasively blast the glassy areas or reapply the resin and sand (time permitting).

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115-1.0	3-1-98	INSPECTION OF ELECTRICAL, MECHANICAL, WATER AND WASTEWATER WORK

FOR RALPH P. SOMMARIVA, Chief
Office of Structure Construction

Volume II

INSPECTION OF ELECTRICAL, MECHANICAL, WATER AND WASTEWATER WORK

It is the Structure Representative's responsibility to ensure that all electrical, mechanical, water and wastewater (EMW&W) work complies with the plans and specifications. However, EMW&W work is a specialized and complex type of work with which the Structure Representative may not be totally familiar.

In order to be assured that the EMW&W work complies with the plans and specifications, the Structure Representative is encouraged to contact the Electrical, Mechanical, Water and Wastewater Branch (Telephone 916-227-8337 calnet 498-8337) to keep them informed of the work progress, to discuss any problems encountered, and to make arrangements to have EMW&W personnel make periodic inspections of the work as it progresses towards completion.

Volume II

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120-2.0	12/01/2001	IMPAIRED CLEARANCE AT TRAFFIC OPENINGS
120-3.0	4/30/1995	FALSEWORK SHEAR VALUES FOR BOLTS/ANCHORS IN CONCRETE
120-4.0	6/13/2003	REVIEWING AND SUBMITTING TEMPORARY SUPPORT (GUYING PLANS) & WORKING DRAWINGS

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

SUBMITTING FALSEWORK DRAWINGS

Directions for submittal of falsework drawings to the Falsework Section of the Office of Structure Construction are included in the Falsework Manual.

The Office of Structure Construction Falsework Section has the following responsibilities relative to falsework:

Falsework Review Unit

A Falsework Review Unit is operational in the Sacramento Office and this unit performs the following falsework functions:

1. Participate in falsework research projects and provide input relative to changes in specifications and policy.

2. Review all falsework drawings and calculations for falsework adjacent to or over railroads and forward the drawings and calculations to the railroad for their review and acceptance. The Falsework Review Unit provides the liaison between the job and the railroad.

3. "Spot Check" falsework drawings and calculations for falsework not adjacent to or over railroads. The drawings to be "spot checked" are selected at random with the objective of ascertaining that current falsework directives and policies are being complied with.

4. Act as consultant to the Structure Representative and provide guidance with complicated falsework problems and resolve questions involving falsework policy.

5. Act as liaison between the Office of Structure Design and Structure Representative to review all drawings and calculations for contractor designed temporary bridges or other temporary facilities which cross a traveled way which is being used by public traffic.

December 5, 1989

07-472404
07-VEN-101-9.0/13.9
F-P101(355)

Kasler Corporation
P. O. Box 1030
Camarillo, CA 93011

Attn: Mr. Tom Meyer
Project Manager

Gentlemen:

The falsework drawings for Camarillo Overhead and Separation (Widen), Bridge No. 52-16, as revised December 1, 1989, have been reviewed and are approved to the extent provided in Section 5-1.02 of the Standard Specifications.

Your attention is directed to your responsibilities pursuant to Sections 5-l.02, 7-1.09, and 51-1.06 of the Standard Specifications as well as the Construction Safety Orders.

You are reminded that the falsework must be constructed to conform to the falsework drawings, that the materials used must be of the quality necessary to sustain the stresses required by the falsework design, and that the workmanship shall be of such quality that the falsework will support the loads imposed on it without excessive settlement or take up beyond that shown on the falsework drawings.

Very truly yours,

R. W. Joe
Sr. Resident Engineer

BY: G. W. Plaas
Structures Representative

CC: Sacto
JCDunn
GAPlaas
file

BRIDGE CONSTRUCTION MEMO 120-2.0

FALSEWORK

December 1, 2001

Sheet 1 of 1

Volume II

IMPAIRED CLEARANCE AT TRAFFIC OPENINGS

The Structure Representative shall advise the Resident Engineer when it is determined that construction operations will temporarily impair the clearance of a structure on the State Highway. The timely notification of temporary impaired clearances due to falsework is essential to preventing potential damage to the structure and injury to the traveling public, the contractor's personnel and Caltrans staff.

The detailed procedures for notification of temporary impaired clearances are contained in Bridge Construction Memo 2-20.0 "Notice of Change in Structure Clearance or Permit Rating" of the Bridge Construction Records and Procedures Manual and Section 3-701F "Falsework Erection and Removal" and Section 3-705 "Public Safety" of the Caltrans Construction Manual.

The notification to the Resident Engineer should give anticipated dimensions of the impaired opening. The anticipated clearance should be calculated from actual field dimension and the contractors approved falsework submittal. The Structure Representatives, when performing the clearance calculations, should include an allowance for; adjustment of falsework grades, changes in pavement elevations, settlement, deflection, etc.

The Structure Representative, immediately after the clearance is impaired, shall verify the dimensions of the impaired opening and notify the Resident Engineer of any necessary revisions to the clearance. The instructions within this section apply to any temporary changes in existing clearances, even if the resulting clearance or permit rating satisfies legal height or load limitations. When the temporary impaired clearance is removed, the Structure Representative shall give the Resident Engineer written notice of the restored clearance.

BRIDGE CONSTRUCTION MEMO 120-3.0

FALSEWORK

April 30, 1995

Volume II

FALSEWORK SHEAR VALUES FOR BOLTS/ANCHORS IN CONCRETE

General Information

It is extremely important to consider the effect on concrete of bolts loaded in shear parallel to the face of the concrete especially within 6 inches of a concrete edge. Examples of these loading conditions are bolts inserted in the edge of decks which may support vertical falsework loads, or for pins which may be placed in decks and loaded horizontally as shear resistance for K-Rail.

Bar reinforcing steel under, and perpendicular to, the axis of a bolt imbedded in an edge of deck provides some resistance to shear failure of the concrete. Bar reinforcing steel under, but parallel to the bolt axis (without perpendicular reinforcing), provides little additional resistance to concrete shear failure. Unreinforced concrete will offer the least resistance to concrete shear failure.

Bolts or anchors inserted into concrete with impact drills or hammers will generally exhibit lower shear resisting capacity because of potential fracturing of the concrete. The lowest shear resisting capacity may well be furnished by wedge fit type anchors in holes made by impact tools since the wedging action can induce additional fracturing of the concrete.

Research

Bolts of 60 ksi tensile strength, approximately 5/8" in diameter and approximately 11-7/8" long, were cast about 7-7/8" deep at various edge distances (d) up to about 6 inches in unreinforced concrete blocks. The bolts were load tested at right angles to the block surfaces after the concrete had gained sufficient strength. Published test results of concrete failure due to the lateral loads on the bolts showed that the modes of failure consisted of concrete failure with and without wedge cones, or with pullout cones, or by shear failure of the bolts.

Test results for concrete shear strengths adjusted to an averaged concrete compressive strength f_c' of approximately 3,000 psi have been plotted in Figure 1. Assuming[2] that the shear strength of concrete is a function of f_c' a family of curves related to the 3,000 psi curve were generated and included in Figure 1. The curves of Figure 1 represent ultimate values which need to be adjusted with an appropriate safety factor which is determined from Table 1.

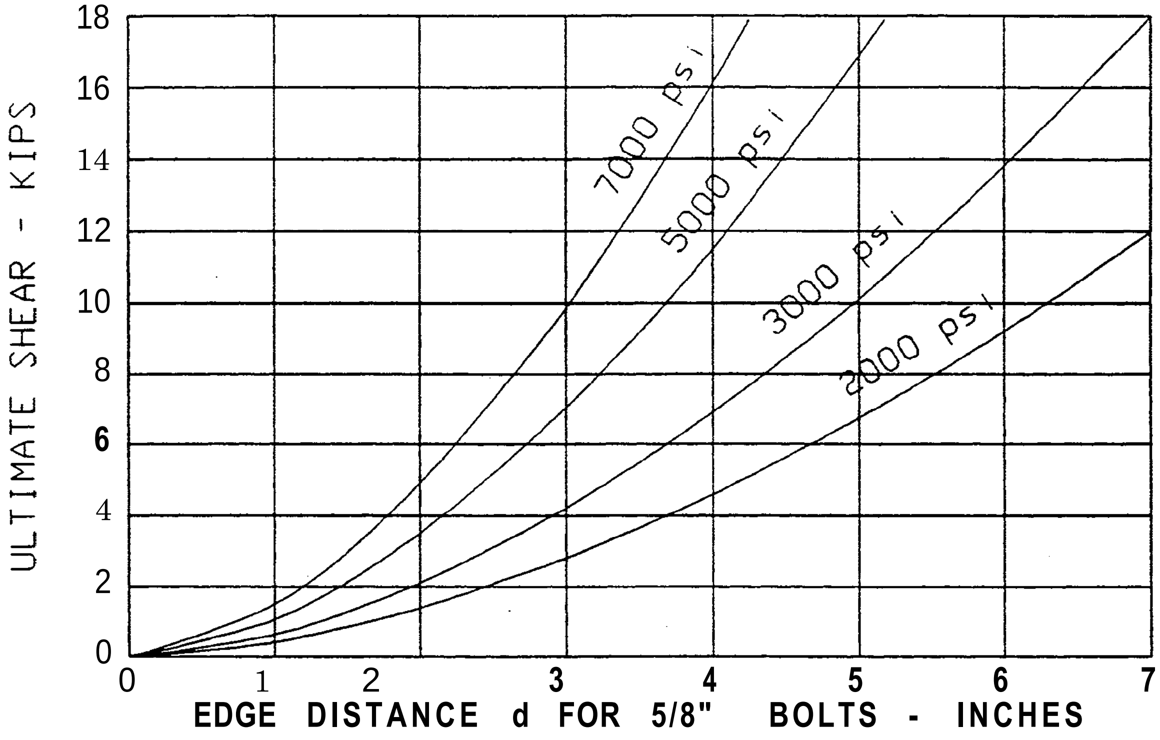


FIGURE 1

One of the most common usages of embedded bolts is depicted in Figure 2. Figure 2 depicts one direction of loading with respect to the concrete surface and shows dimension d which represents the distance from the edge of the concrete to the center line of the bolt.

Permitted Use

Determine the distance the loaded bolt will be from the loaded edge 1 or change in section, of the concrete, The term 'bolt' shall be meant to include inserts, rods, or other similar devices. Haunched concrete sections similar to the underside of bridge deck overhang sections, should not be given additional value. Select a concrete shear strength related to an appropriate ultimate concrete strength curve from Figure 1. Divide the selected ultimate concrete shear strength value by an appropriate safety factor as determined from Table 1.

Table 1 lists minimum safety factor values to be used for cast-in-place bolts. A minimum safety factor of 2 may be used where reinforcing is located normal to the axis of the bolt in the concrete shear loaded zone provided the reinforcing is located between the concrete face and the midpoint of the embedded bolt length. This reinforcing must extend through the concrete failure zone (See Figure 2) to sound concrete. Otherwise use a safety factor of 2.25 or higher.

A minimum safety factor of 2.75 may be used when reinforcing steel located parallel to the bolt axis (without reinforcing normal to the bolt axis) will be within the shear loaded zone and will have a length reaching to at least the midpoint of the embedded bolt. If no parallel reinforcing will be within a shear loaded zone use the higher safety factor of 3.0.

SAFETY FACTORS:

REINFORCING TYPE	CAST-IN-PLACE BOLT	BOLT INSERT
Bar(s) within 2" of the concrete face located normal to the bolt axis on the loaded side of the bolt.	2.0 - 2.5	2.25 - 2.50
Bars parallel to the bolt axis on the loaded side of the bolt (no normal reinforcing).	2.75 - 3.0	2.75 - 3.0
No reinforcing on the loaded side of the bolt.	3.0	3.0
Torque tightened bolts, regardless of location of reinforcing.	3.0	3.0

TABLE 1

Shear loaded zones are depicted in Figure 2 and are described as follows:

Cone Pullout - A cone with the apex situated behind the embedded end of the bolt with the center line of the cone on the axis of the bolt and with sides sloping a minimum of 45° away from the bolt axis.

Trapezoidal Wedge - A trapezoid volume with the base on the concrete surface a distance d from the bolt center line, having sides sloping inward at 45° angles meeting a plane of length equal to the bolt diameter at the opposite side of the bolt, and with this area having a volumetric length equal to the length of the embedded portion of the bolt.

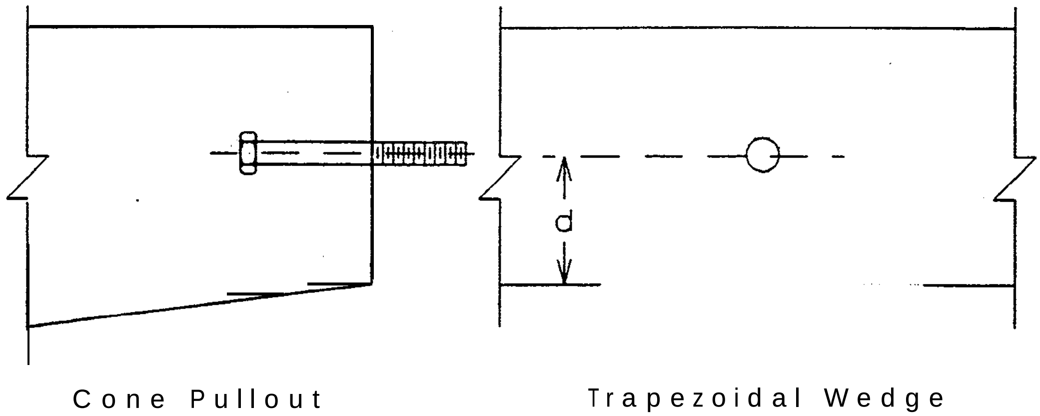


FIGURE 2

When information about reinforcing on the loaded side of a bolt cannot be verified the highest safety factor will be used. No increase or reduction in the safety factor value, will be given to bolts with embedded bolt heads.

Figure 3 illustrates a single bolt placed in a deck overhang. Bolts shall not be spaced closer than 8 times the d dimension or 3.5 times the sum of the embedment lengths of adjacent bolts whichever is larger. The 8 times the d dimension is a recommendation made in the published test results. The 3.5 (two times the tangent of 60°) times the sum of the embedment lengths is based on evidence that the concrete can fracture or fail in shear at an angle of 60° to the axis of the bolt.

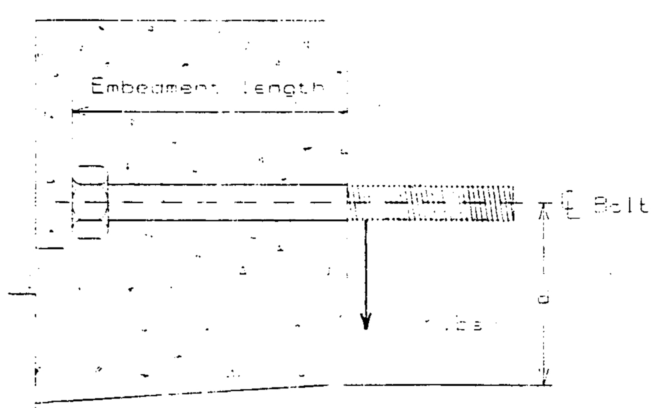


FIGURE 3

Since one of four types of failure modes occurred during testing no adjustments need be made for various bolt sizes up to 5/8" in diameter. For bolts larger than 5/8" diameter use dimension d as being equal to the distance from the edge of the concrete to the nearest portion of the bolt hole (In Figure 3 this dimension would be d minus one-half the bolt diameter).

It is always a good idea to test load a typical section of falsework supported by bolts or inserts to twice the anticipated loading at a safe location.

Example:

Assume 5,000 psi concrete in a deck overhang with no bottom mat reinforcing for which holes for 5/8" bolts are to be drilled 3.75 inches from the soffit with air tools for torque type bolt inserts that are to be used for supporting a falsework system for removal of concrete railing. Determine bolt capacity and minimum bolt spacing for 5 inches of maximum embedment.

From Figure 1 the ultimate value for shear failure may be selected as 10.1 Kips, and from Table 1 the safety factor value should be no less than 3.0.

Assumed safe working load for the bolt = 10.1/3.0 = 3.4 Kips. Minimum bolt spacing is the larger of 8(d) or 3.5(5" + 5"):

8(3.75) = 30.0 Inches
3.5(5"+ 5") = 35.0 Inches + This governs

References:

1. Ueda,T.; Stitmannaithum,B.; and Matupayont,S., "Experimental Investigation on Shear Strength of Bolt Anchorage Group," ACI Structural Journal, V. 88, No. 3, May-June 1991, pp.2920300.

2. ACI Committee 349, "Proposed Addition to: Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)" and "Addition to Commentary on Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-76)," ACI Journal, *Proceedings* V. 75, No. 8, Aug. 1978, pp. 329-347,

Additional Information

Consult the Bridge Construction Records and Procedures Manual Memo 135-5.0 and Memo 168-2.0 regarding bolts in concrete.
Additional information on concrete anchorage devices may be found in Section 75-1.03 of the Standard Specifications. While these references pertain to permanent installations the guidelines are worthy of note.
Installation for temporary work do not require Translab approval. It is important however, that manufacturer's recommendations be followed except where there will be obvious deviations from this memo.

Volume II

REVIEWING AND SUBMITTING TEMPORARY SUPPORT (GUYING PLANS) & WORKING DRAWINGS

Temporary support (guying) plans submitted by the Contractor must be reviewed by the Offices of Structure Construction staff thoroughly and independently. You may find the following references useful during this review:

- Standard Specifications:
  Section 52-1.07, 'Placing'
  Section 7-1.09, 'Public Safety'

- Falsework Manual:
  Section 4-5, 'Cable Bracing Systems'
  Falsework Memo No. 9, 'Short Poured-In-Place Concrete Piles to Resist Uplift and Lateral Loads'.

- Contract specific Special Provisions

The minimum horizontal load to be allowed for wind on steel cages shall be the sum of the products of the wind impact area and the applicable wind pressure value for each height zone. The wind impact area is the total projected area of the cage. Wind pressure values shall be determined from the following table:

HEIGHT ZONE (METERS ABOVE GROUND)	WIND PRESSURE VALUE (Pa)
0-9	960
9-15	1200
15-30	1440
Over 30	1675

Temporary support plans shall include sufficient detail to ensure stability of reinforcing cages during all phases of construction. The plan shall also include provisions for keeping stable the column cage and forms during the transition from one stage to the next as well as a list of all equipment to be utilized to handle the erection.

Copies of working drawings and design calculations per Section 52-1.07, 'Placing' of the Standard Specifications shall be submitted to the Offices of Structure Construction HQ in Sacramento after approval by the Structure Representative.

When such plans as described by the previous paragraph are adjacent to railroads, the procedure for approving these plans will be in accordance with the requirements for submitting falsework when railroad companies are involved. A complete explanation of this procedure may be found in the Falsework Manual Section 2-1.06B, ‘Procedure when Railroad Company Approval is Required’.

BRIDGE CONSTRUCTION MEMO 122-0.0

SHORING

June 1, 1997

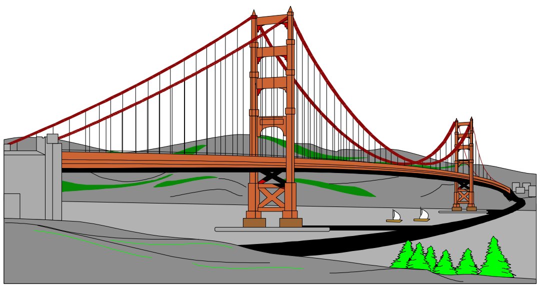
Sheet 1 of 1

Volume II

TABLE OF CONTENTS FOR SECTION No. 122

Memo No.	Date	Title
122-1.0	12-05-89	SUBMITTING SHORING PLANS

RALPH P. SOMMARIVA, Chief
Office of Structure Construction



BRIDGE CONSTRUCTION MEMO 122-1.0

SHORING

June 1, 1997

Sheet 1 of 2

Volume II

SUBMITTING SHORING PLANS

General Information

Review of shoring plans shall be in accordance with the guidelines and policy set forth in the Office of Structure Construction Trenching and Shoring Manual.

Shoring Which is Not Adjacent to or Under a Railroad

When the Structure Representative has completed reviewing the shoring plans and calculations, and has determined that the shoring plans meet specification requirements, a letter shall be sent to the Contractor stating that the drawings have been reviewed and approved. This letter authorizes the Contractor to begin constructing shoring. Note that the Contractor must not begin shoring construction until such time as this letter has been issued to the Contractor. The letter should state that the approval is to the extend provided under Section 5-1.02 of the Standard Specifications. This letter should also direct the Contractor's attention to his/her responsibilities under Sections 5-1.02 of the Standard Specifications. It should remind the Contractor of the requirement that the shoring must conform to the shoring plans, that the materials used must be of the quality necessary to sustain the stresses required by the shoring design, and that the workmanship shall be of such quality that the shoring will support the loads imposed on it without excessive movement (See Attachment No. 1 for an example of a properly prepared shoring approval letter.)

For shoring that is not adjacent to or under a railroad, it is mandatory that one copy of the shoring plans, and one copy of the Structure Representatives calculations be submitted to the Sacramento Office of Structure Construction, immediately after they have been reviewed and approved by the Structure Representative.

A letter of transmittal must accompany all shoring plans and calculations that are submitted to the Office of Structure Construction Falsework Review Unit. Include a copy of the approval letter that was issued to the Contractor.

The Structure Representative should retain one set of the approved shoring plans and calculations in the job files.

Shoring Adjacent to or Under Railroads

When shoring is adjacent to or crosses under a railroad, additional requirements must be complied with. Any shoring within 15' of the centerline of tracks is considered adjacent.

The Structure Representative will check the shoring plans, and if necessary, return them to the Contractor for correction. After shoring plans and calculations have been reviewed by the Structure Representative and he/she is satisfied that the shoring plans meet specifications, the following items are to be sent to the Office of Structure Construction.

(1) Five copies of Contractor's shoring plans if the Union Pacific (Southern Pacific), Burlington Northern Sante Fe or Western Pacific railroad are involved.

(2) Three copies of Manufacturers' data relative to manufactured devices.

(3) Three copies of Contractor's calculations.

(4) Three copies of the Structure Representative's calculations.

Note: One copy of the above is for the Sacramento Office of Structure Construction's use, and the other copies are forwarded to the railroad by the Office of Structure Construction. In the event that railroad personnel at the job site need copies of the above information, they are to obtain it from their headquarters.

When the above noted data is submitted to the Office of Structure Construction, it shall be accompanied by a letter of transmittal in which the Structure Representative will list the information submitted, and state that the shoring plans and calculations have been reviewed and that they are considered to be satisfactory. The Office of Structure Construction will review this data. The Structure Representative should not stamp the shoring plans 'Approved' until they have been notified that the railroad has reviewed and accepted the shoring plans.

Incomplete or unsatisfactory data will be returned to the Structure Representative for correction. Upon confirming that the plans and calculations are complete and satisfactory, the information will be forwarded to the railroad for their review and acceptance.

When the Sacramento Office of Structure Construction and railroad reviews are complete, and the Sacramento Office of Structure Construction advises the Structure Representative that the railroad considers the shoring plans to be satisfactory, the Structure Representative will send a letter to the Contractor stating that the plans have been reviewed and approved. This letter authorizes the Contractor to begin constructing the shoring. Note that the Contractor <u>must not begin shoring</u> construction until such time as the approval letter has been issued to the Contractor. (See Attachment No. 1 for an example of a properly prepared shoring approval letter.)

<u>Shoring Review Unit</u>

A Shoring Review Unit will be operational in the Sacramento Office and this unit will perform the following functions:

1. Participate in shoring research projects and provide input relative to changes in specifications and policy.

2. Review all shoring plans and calculations for shoring adjacent to or under railroads and forward the plans and calculations to the railroad for their review and approval. The Shoring Review Unit provides the liaison between the job and the railroad.

3. "Spot Check" shoring plans and calculations for shoring not adjacent to or under railroads. The plans to be "spot checked" are selected at random with the objective of ascertaining that current shoring directives and policies are being complied with.

4. Act as consultant to the Structure Representative and provide guidance with complicated shoring problems and resolve questions involving shoring policy.

STATE OF CALIFORNIA-BUSINESS, TRANSPORTATION AND HOUSING AGENCY PETE WILSON, Governor

DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
OFFICE OF STRUCTURE CONSTRUCTION MS-9
P.O. Box 942874
SACRAMENTO, CA 94274-0001

(916) 227-7777
(916) 227-8835
FAX (916) 227-8179

June 1, 1997

Co-Rte-PM
00-000104

Sample Construction Co.
Sample Lane
Sample, CA 00000-0000

Gentlemen:

The shoring plans for the temporary retainment of soil and embankment material at Abutment 1 of General Avenue Overcrossing, Bridge No. 2B-X4X, dated August 14, 1989, have been reviewed and are approved to the extent provided in Section 5-1.02 of the Standard Specifications. [Add the following when using the contractor's/consultant's soil parameters; This approval is based upon the soil parameters submitted by your consultant.] [Add when needed for trench shields; Note that your submittal did not include calculations from the trench shield design engineer. It is therefore understood that your licensed engineer has verified that the shields themselves are capable of sustaining the loads allowed by the shield manufacturer.]

Your attention is directed to your responsibilities pursuant to Section 5-1.02 and 7-1.09, of the Standard Specifications as well as the Construction Safety Orders.

You are reminded that the shoring must be constructed to conform to the shoring drawings, that the materials used must be of the quality necessary to sustain the stresses required by the shoring design, and that the workmanship shall be of such quality that the shoring will support the loads imposed on it without excessive movement.

Sincerely,

Y.Y. Resneer
Resident Engineer

by

W.W. Strurep
Structure Representative

c:

<table>
<tr><td colspan="2">OFFICE OF STRUCTURE CONSTRUCTION
Bridge Construction Records and Procedures Manual 02-09</td></tr>
<tr><td>BRIDGE CONSTRUCTION BULLETIN

Approved: <u>Original Signed by Dolores Valls</u>
Dolores Valls, Deputy Division Chief
Offices of Structure Construction</td><td>File: BCM 122-2
SHORING

Date: October 15, 2002
Expires: July 1, 2003
Supersedes: BCM 122-2 Dated May 19, 1997</td></tr>
</table>

Subject: Support from Roadway Geotechnical Engineering

Shoring systems requiring the use of slope stability (i.e. excavation exceeding the Cal OSHA sloping or benching requirements) should be reviewed by the Roadway Geotechnical Engineering unit for adequacy. To obtain assistance from the Roadway Geotechnical Engineering unit, contact the appropriate Geotechnical Design Branch (North, South, West) that your project is in using the following link:

http://onramp.dot.ca.gov/hq/esc/geotechnicalservices/design.htm

cc: BCR&P Manual Holders
R. Pieplow, HQ Const.
Consultant Firms
J. Van Velsor, Geotech.Services

BRIDGE CONSTRUCTION BULLETIN

Approved:___Original signed by__________
DOLORES VALLS, Deputy Division Chief
Offices of Structure Construction

File: BCM 122-3
SHORING

Date: December 21, 2001
Expires: None
Supersedes: None

Subject: Torvane Usage for Sloped Excavations

The Offices of Structure Construction has received and approved a contractor's standard shoring plan utilizing a Torvane (or shearvane) device. The Torvane is a soil-testing tool utilized for rapid determination of cohesive soils' undrained shear strength (Su). The use of Torvane is an acceptable practice according to CalOSHA for determining the maximum allowable slope of an excavation.

The following soil properties were assumed for the evaluation of such a standard plan. If your soil properties differ significantly, additional analysis will be required:

? = 120 pcf (1920 kg/m^3)
? = 25°

Projects utilizing a Torvane will additionally require the following conditions be met and addressed with each shoring submittal:

- Tension cracks must be prevented from developing at top of excavation.
- Water must be prevented from ponding at the top of and toe of the slope.
- Soil must be consistent and homogeneous throughout the excavation.
- CalOSHA requirements must be adhered to at all stages of the excavation.

In situations where the contractor is proposing to use a Torvane, please consult the Trenching and Shoring Senior Specialist in the HQ Offices of Structure Construction at (916) 227-8060 for assistance.

c: BCR&P Manual Holders
CMB - Consultant Firms
RPieplow, HQ Division of Construction

BRIDGE CONSTRUCTION BULLETIN **Approved:** ______________ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 124-2** **D E M O L I T I O N** **Date: January 2, 1998** **Expires: January 1, 1999** **Supersedes: None**

Subject: Demolition Plan Review

This bulletin has been developed to assist field personnel responsible for the administration and review of bridge demolition plans. It is intended to address the current field issues regarding review of demolition plans and the quality assurance measures to be employed when reviewing demolition operations. The review and processing of the demolition plan should be documented thoroughly.

One copy of the approved drawings, one copy of the engineer's calculations, and the Structure Representative's calculations are to be submitted to the Sacramento office of Structure Construction, along with a copy of the approval letter sent to the contractor.

During the review, if it becomes apparent additional assistance is required due to the complexity of the structural analysis; the responsible design unit (OSD, CCMB, EFPB) will provide assistance. Before requesting assistance, discuss the plan with the Bridge Construction Engineer (Senior Bridge. Engineer) responsible for your project to obtain agreement that assistance is required. Once agreement is attained, contact the Design Senior or Project Manager responsible for your project and request assistance. A written request may be required depending on the complexity of the analysis.

The attachments are intended to be a guideline and not an all-inclusive list for bridge removal review. Bridge removal is a very important item to inspect for safety concerns regarding the public and contract personnel. Safety always comes first and if something does not look appropriate, do not compromise safety.

If further assistance is needed, contact the appropriate Sacramento OSC HQ Staff Engineer or Senior assigned to your district.

Attachment

cc: BCR&P Manual Holders
Consultant Firms
BGauger, Construction Program Manager

BRIDGE DEMOLITION PLAN REVIEWS

1. Recommend the Contractor investigate the as-built conditions of the structure and determine if special conditions exist (column pins and deck hinge(s) that may need to be restrained during bridge removal operations).

   A. Designer should include as-built plans for the structure to be removed in the RE Pending file, or the Structure Representative should request these upon first review of the RE Pending file.
   B. The Contractor should obtain as-builts from the district in which the work is situated per Standard Specifications.

2. Inform the Contractor to submit a complete bridge removal/demolition plan, prepared by an engineer who is registered as a Civil Engineer in the State of California, at least ten days prior to commencing demolition unless the Special Provisions state otherwise. This plan should detail the procedures and sequence for removing portions of bridge(s), including all features necessary to remove the bridge(s) in a safe and controlled manner. The stability of the structure must be demonstrated by calculations at each stage of removal. (This section does not apply when bridge removal is only specified for bridge railing work and/or widening work that requires small amounts of bridge removal.)

3. Remind the contractor that any vertical shoring used to stabilize the structure during removal operations shall conform to the provisions set forth in SS 51-1.06 "Falsework" of the Standard Specifications (unless otherwise noted in the Special Provisions).

4. If the contractor is removing a portion of a bridge (e.g. barrier rail only, edge of deck for a widening), the removal operations shall be performed without damage to any portion of the structure that is to remain in place. In such cases, tools with a manufacturer's rated striking energy in excess of 1625 J (1,200 foot pounds) per blow shall not be used for breaking or removing concrete which is attached to or supported by the bridge. (SS 15. 4.02)

PRE-JOB MEETING

Discuss the following items at the pre-job meeting:

-Standard Specifications (SS)
-Special Provisions (SP)
-Cal-OSHA Construction Safety Orders
-Falsework Manual (if necessary)

The Standard Specifications requires the Contractor to submit a demolition plan at least ten days prior to the beginning of bridge removal work. However, due to the complexity of a structure, the Special Provision may allow additional time.

INITIAL REVIEW

Perform an initial review and check for contract compliance. The plan must be generally suitable for the site conditions to be encountered and should provide adequate information clearly outlining the removal procedure proposed by the Contractor. The following are some basic items to check before your formal review.

1. Review the details and proposed staging of the removal operation. The plan should show the methods and sequence of removal. (SS 15-4)

2. Verify that the bridge removal plan is prepared by an engineer who is registered as a Civil Engineer in the State of California.

3. Calculations for vertical shoring, restraining systems for columns and deck hinges should be included, if applicable.

4. The design calculations shall be adequate to demonstrate the stability of the structure during all stages of the removal operations.

5. A complete, Contractor's equipment list with attachments, dimensions, axle loadings and equipment weights should be shown along with proposed placement locations to check for possible structure overloads. (SS 7-1.02)

6. Timeline, milestones, and contingency plans need to be included when dealing with traffic or other potential safety problems, traffic delays, and construction windows.

7. Protection of existing utilities and non-highway facilities should be covered. (SS 7-1.11 "Preservation of Property").

8. The locations of temporary hand railing and barrier railing for bridge decks should be shown for staged bridge removal. This detail is necessary for the protection of the employees and the public.

FORMAL REVIEW:

1. The safe work areas for the contractor's personnel, and Caltrans personnel should be shown on the demolition plan. The protection of public traffic (vehicles and/or pedestrians) and private property must be covered thoroughly in the demolition plan, including the safe routing of public traffic during demolition operations.

2. The location of the demolition equipment and the method in which the equipment is used to remove the structure should be reviewed. This may assist you in evaluating the stability of the structure under live loads during bridge removal. This is extremely important when a portion of the structure is to remain standing and equipment is to stay on the structure.

3. A freely falling mass or a falling mass attached to a cable; rope or chain shall not be used above or within 9m (30 feet) horizontally of any area open to the public unless adequate protective shields are in place (SS 15.4.02), and/or unless otherwise noted in the Special Provisions

4. The impact and possible damage due to excessive demolition debris falling and collecting on roadway section, adjacent structure, supporting falsework, protective cover, or even structural elements supporting the structure being removed need to be investigated. Again, the equipment loads should be considered in combination with debris build up. If needed, debris shields shall be used to protect the surrounding area, structural elements, and falsework and shall be detailed and shown on the demolition plan. Also, ensure there is a provision on the plans for dust control.

5. A timeline should be noted for each operation if traffic control is involved. When working within a "time-window", progress should be monitored against planned progress milestones and a contingency plan should be agreed upon prior to the start of removal operations if work falls behind planned progress.

6. The bridge removal plan shall conform to the requirements in Section 5-1.02, "Plans and Working Drawings," of the Standard Specifications. The number of sets of drawings and design calculations and times for review for any bridge removal plans shall be the same as specified for falsework drawings in Section 51.l.O6A, "Falsework Design and Drawings," of the Standard Specifications. (Unless otherwise specified in the Special Provisions.)

7. One copy of the approved drawings, the engineer's calculations, and the Structure Representative's calculations are to be submitted to the Sacramento office of Structure Construction, along with a copy of the approval letter sent to the contractor.

OTHER AGENCY REQUIREMENTS

1. The Contractor shall conform to all local sound, water, noise, and air pollution control requirements per Standard Specification Section 7, Special Provisions, and/or applicable permits.

2. The Contractor must get a disposal permit for disposal of material outside the highway right of way per Standard Specification Section 7- 1.13.

3. A Cal-OSHA permit is required for the demolition of any structure greater than 10.9 m (36 feet) in height. (Cal-OSHA Article 2 Section 341, "Permit Requirements")

4. If railroad review of the demolition plan is required on your contract, make sure to inform the agency well in advance of the work, especially if the demolition work is a critical item on the project schedule. Information regarding working with Railroads can usually be found in the Special Provisions Section, "Relations with Railroad Company". Also, inform the Contractor at the pre-job meeting regarding the possible time delay by impromptu plan submittal and revisions. The procedure for approving these plans is identical to that used for falsework plan approval. The instructions can be found in the Falsework Manual Section 2-1.06B 'Procedure when Railroad Company Approval is Required.'

BRIDGE CONSTRUCTION BULLETIN

Approved:_original signed by R.P.Sommariva
R. P. SOMMARIVA, Chief
Office of Structure Construction

File: BCM 124-3
DEMOLITION

Date: May 19, 1999
Expires: May 19, 2000
Supersedes: None

Subject: Demolition Over and/or Adjacent to Union Pacific Railroad Company Tracks

Railroad Guidelines

To expedite the review process of demolition plans by the Union Pacific Railroad Company (UPRR), it is advisable that the drawings submitted by the contractor adhere to the requirements of Union Pacific. The latest railroad's requirements are titled *GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD*. Refer to Attachment No. 1 for a copy of this guideline.

The contract special provisions will list the clearance requirements measured from the centerline of the railroad tracks. If clearances are not included in your contract documents, refer to UPRR Std. Dwg. 0035, "Barriers and Clearances to be Provided at Highway, Street, and Pedestrian Overpasses" for minimum construction clearance requirements. Refer to Attachment No. 2 for a copy of this drawing. This drawing shows the latest UPRR clearance requirements and will be incorporated into future contracts.

Where there is a conflict between the contract specifications and the guidelines issued by the railroad, the contract specifications shall prevail.

Railroad Requirements

The UPRR has requested that drawings accompanying demolition plans be submitted on 11"x17" (279.4 mm x 431.8 mm) sized paper. Future special provisions will be revised to state that the drawings for the railroad should be on 11"x17" (279.4 mm x 431.8 mm) sized paper. Until this request becomes a specification requirement, you can request that the contractor submit the three sets of drawings accompanying demolition plans for railroad review be on 11"x17" (279.4 mm x 431.8 mm) sized paper.

Some common requirements are often overlooked and have resulted in submittals being returned by the railroad. The demolition plans should state that all demolition will comply with the latest railroad demolition guidelines. The demolition plans should note how the contractor will gain access to the site, particularly if they must cross the railroad tracks. Track protection details are shown in the aforementioned UPRR guidelines, and details must be included on the demolition plans.

The demolition plans should note if there are any existing drainage ditches or access roads being affected by the contractor's operations. If there are no existing drainage facilities or access roads, the demolition drawings should note this fact. Keep in mind that personnel from the railroad who are unfamiliar with the site often review the demolition plans.

The above railroad requirements should be discussed at the pre-construction meeting with the contractor. It should also be stated that approval of demolition plans over and/or adjacent to UPRR tracks will be contingent upon UPRR approving the plans.

Distribution of Demolition Plans

The Structure Representative will check the demolition plans, and if necessary, return them to the Contractor for correction. After the demolition plans and calculations have been reviewed by the Structure Representative and he/she is satisfied that the demolition plans meet the specification requirements, the following items are to be sent to the Office of Structure Construction Headquarters (OSC HQ):

1. Four copies of Contractor's demolition plans (a minimum of three sets of 11x17 drawings for the railroad is preferred)

2. Three copies of the Contractor's calculations, tabbed to show key elements affecting the demolition over and adjacent to the railroad company's tracks

3. Three copies of Structure Representative’s calculations, tabbed to show key elements affecting the demolition over and adjacent to the railroad company’s tracks

4. Three copies of manufacturer’s data relative to manufactured devices

Note: One copy of the above is for the OSC HQ office use, and the other copies are forwarded to the railroad. In the event that railroad personnel at the job site need copies of the above information, they are to obtain it from their headquarters.

In order to complete the demolition review within the contract time specified, the Structure Representative should expedite their review and forward the submittal to the OSC HQ (Attention: John Gillis) via overnight mail.

When the above noted data are submitted to the OSC HQ office, a letter of transmittal from the Structure Representative shall accompany them. The transmittal letter shall list the information submitted and state the demolition plans and calculations have been reviewed and that they are considered to be satisfactory. The Structure Representative should not stamp the demolition plans 'Approved' until OSC HQ has notified them that the railroad has reviewed and accepted the demolition plans.

Railroad Review and Approval

Incomplete or unsatisfactory data will be returned to the Structure Representative for correction. The OSC HQ will review this data. Upon confirming that the plans and calculations are complete and satisfactory, the information will be forwarded to the railroad via overnight mail for their review and acceptance.

Please note that all correspondence with the railroad regarding the status of submittals under their review should be directed to John Gillis. At the railroad’s request, in no case should you contact the railroad directly.

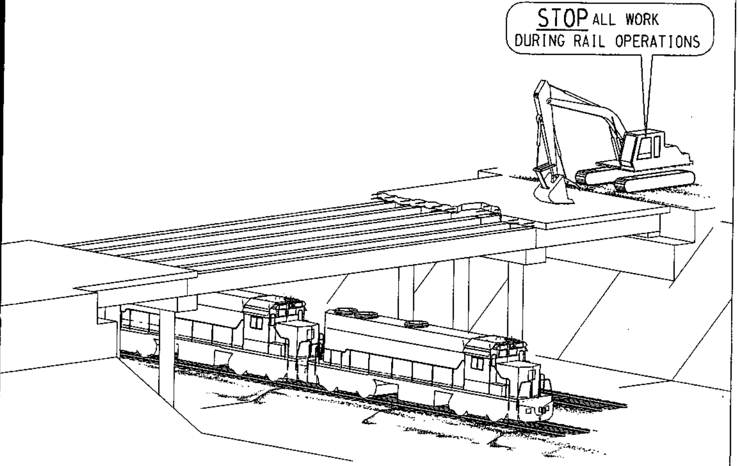
When the railroad completes their review and finds the plans to be acceptable, they will advise the OSC HQ who in turn will advise the Structure Representative that the railroad considers the demolition plans to be satisfactory. The Structure

Representative will then stamp the plans 'approved' and send a letter to the Contractor stating that the plans have been reviewed and approved. Assuming proper notification has been made to the UPRR that their horizontal and vertical clearances will be impaired and that a flagger is required, the Contractor may begin demolition work. Note that the Contractor shall not begin any demolition within the railroad right-of-way until such time as the approval letter has been issued to the Contractor.

Attachments

c: BCR&P Manual Holders
Consultant Firms
CABomar, Chief, Railroad Agreements Section
BFelker, Construction Program Manager

GUIDELINES FOR PREPARATION OF A BRIDGE DEMOLITION AND REMOVAL PLAN FOR STRUCTURES OVER RAILROAD



UNION PACIFIC RAILROAD

OFFICE OF CHIEF ENGINEER DESIGN
1416 DODGE ST.
OMAHA, NE 68179

BRIDGE CONSTRUCTION MEMO 124-3
ATTACHMENT No. 1 (5/99)
Sheet 1 of 13

INDEX

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I. GENERAL

A. The Contractor's work shall in no way impede the train operations of the Union Pacific Railroad.

B. The Contractor shall develop a work plan assuming that minimal track windows will be available.

C. The Contractor shall be responsible for planning and executing all procedures necessary to remove the overhead bridge in a safe and controlled manner.

D. The Railroad's tracks and property shall be protected at all times.

E. The contractor shall ensure the area immediately adjacent to operational tracks shall remain free from stumble or like hazards to the ground Railroad personnel to prevent injuries. Open excavations shall be in accordance with current CE Drawing 106613 and shall be protected by appropriate fencing.

F. The words "demolition" and "removal" will be used interchangeably.

G. All removed materials shall be disposed of outside the Railroad right-of-way at no expense to the Railroad.

H. No work is allowed within 50 feet of the nearest rail when trains pass the work site.

I. Staged demolition of the portions of structure immediately adjacent to operational tracks will not jeopardize the integrity of the structure over said tracks until actual removal of the portion of the structure over the tracks is being done.

J. A flagman is required when any work is performed within 25 feet of the nearest rail.

K. No blasting will be permitted on Railroad's right-of-way.

II. BRIDGE REMOVAL PLAN

A. The Contractor shall submit a complete Bridge Removal Plan to the Railroad. The Bridge Removal Plan shall include details, procedures and the sequence of staged removal of the bridge, including all steps necessary to remove the bridge in a safe and controlled manner.

B. The Contractor shall submit to the Railroad; three (3) complete sets of the Bridge Removal Plan for review and comments. The Plan shall be sealed by a Civil or Structural Engineer registered in the state where the proposed demolition will take place. A minimum of three (3) weeks shall be allowed for the Railroad's review after the complete submittal is received. No removal operations will be permitted over the Railroad right of way until the submitted material has been reviewed and comments provided.

C. Review and comment of the Removal Plan by the Railroad will not relieve the Contractor of the ultimate responsibility and liability for the demolition of the structure.

D. The Removal Plan shall include the following:

1) Plan, elevation and location of the bridge, and the locations of any access roads needed for movement of the equipment. The as-built drawings may be used for the submittal provided the removal steps are clearly marked and legible.

2) Indicate the position of all railroad tracks below the bridge and identify each track as mainline, siding, spur, etc.

3) Bridge removal sequence and procedures for entire bridge including the staging for the removal of the superstructure and substructure.

4) List type and number of equipment required and their locations during demolition operations.

5) Locations and types of temporary supports, shoring or bracing required. These members shall be designed to meet Union Pacific Railroad current standard drawing 106613 "General Shoring Requirements", "Guidelines for Design and Construction of Falsework for Structures Over Union Pacific Railroad", "Guidelines for Design and Construction of Shoring Adjacent to Active Railroad Tracks", and the appropriate local and national building and design code requirements.

6) The proposed vertical and horizontal clearance from all tracks to the temporary and permanent supports. The minimum vertical and horizontal clearances shall be as per attached frame protection details.

7) If any temporary supports interfere with the natural drainage along the Railroad right-of-way, a temporary drainage plan shall be submitted for review and comment prior to constructing temporary supports. The proposed drainage plan shall route all drainage away from the railroad tracks.

8) Details, limits, and locations of protective covers or other measures proposed to be used to protect the tracks. This includes any shields or other measures that will protect the tracks from falling debris during removal of the overhead bridge and from any debris rolling down the side slopes or otherwise coming into the area around the tracks which could affect train operations. Design loads, including impact loads, shall be noted. In addition equipment should be on site capable of removing debris and track shield from operational tracks.

9) All procedures necessary to remove the bridge in a safe and controlled manner. The estimated time for complete removal over the tracks shall be noted.

10) All overhead and underground utilities in the area affected by removal of the bridge shall be located on the drawings, including any fiber optic, railroad signal, and communication lines.

11) The location and details of track crossings required for moving of the equipment across the railroad tracks.

12) Limits of demolition of substructures.

13) Details of on-site fire suppression.

III. PROCEDURE

A. During removal operations the remaining structure shall be stable during all stages of the removal operations.

B. Prior to proceeding with bridge removal the sealing Civil or Structural Engineer, or his authorized representative working for the Contractor, shall inspect the temporary support shoring, including temporary bracing and protective coverings, for conformity with the working drawings. The Engineer shall certify in writing to the Railroad that the work is in conformance with the drawings and that the materials and workmanship are satisfactory. A copy of this certification shall be available at the site of work at all times.

C. Coordinate the removal schedule with the Railroad. All the removal work within the track area shall be performed during the time windows when the trains are not passing the work site.

D. All substructures shall be removed to at least 3 feet below the final finished grade or at least 2 feet below base of rail whichever is lower, unless otherwise specified by the Railroad.

E. All debris and refuse resulting from the work shall be removed from the right of way by the contractor and the premises left in a neat and presentable condition.

F. The work progress shall be reviewed and logged by the Contractor's Engineer. Should an unplanned event occur, the Contractor shall inform the Railroad and submit procedure to correct or remedy the occurrence.

G. Preferably all demolition and beam removal shall be from above. In the case that the beams require removal from below, the beams may temporarily straddle the tracks. The following steps shall be taken:

1) The work shall be scheduled with the Railroad's Service Unit Superintendent subject to the Railroad's operational requirements for continuos train operations. The beams removed in sufficient time for train passage.

2) The tracks shall be protected and no equipment placed on the tracks.

3) The beams shall be blocked and not come in contact with the tracks. Blocking shall not be placed on the tracks.

4) The beams and all equipment will be moved a minimum of 15 feet from the nearest rail of the tracks when a train is passing.

IV. TRACK PROTECTION

A. The track protective cover shall be constructed before beginning bridge removal work and may be supported by falsework or members of the existing structure. See the attached Track Shield Detail and Frame Protection Detail for additional requirements. Types of protective covers that may be acceptable methods for protecting the tracks are:

1) A decking supported by the bridge or a suspended cover from the bridge above the track clearance envelope.

2) A track shield cover over the tracks per the attached detail.

3) A framed cover outside the track clearance envelope.

4) A catcher box or loader bucket under decking and parapets overhanging the exterior girders.

B. Construction equipment shall not be placed on the tracks unless tracks are protected.

C. Temporary haul road crossings shall are of either Section Timbers or Precast Concrete Panels. The type of crossing shall be determined by the Manager of Industry and Public Projects. Solid timbers or ballast with timber headers shall be used between multiple tracks. If temporary crossing is accessible to public crossing shall be protected with barricades or locked gates when contractor is not actively working at the site or weekends.

D. Track protection is required for all equipment including rubber tired equipment operating within 25 ft. or over the tracks.

V. CRANES

A. When cranes are operated near the tracks the following is required:

1) Only cranes with the capacity to handle the loads may be used. Front end loaders and backhoes cannot be used to lift over the tracks.

2) The Contractor shall verify that the foundations under the crane can support the loads.

3) The size and material type of crane mats shall be submitted to the Railroad for review and comment. No mat substitution will be allowed. The mats shall be rigid and of sufficient capacity to distribute the crane loads and prevent tipping of the crane.

4) Installation of temporary track crossings for equipment shall be scheduled with the Manager of Industry and Public Projects .

5) Additional track protection is required when crossing with a crane. The protection methods shall be submitted to the Railroad for review and comment.

6) Equipment shall not place outriggers on the tracks or ballast.

7) Cranes shall not be placed within the track clearance envelope without flagman protection.

VI. CUTTING TORCHES

A. When a cutting torch is used near the tracks or any timber, the following steps shall be taken:

1) Fire suppression equipment is required on-site.

2) Do not use a torch over, between, or adjacent to the tracks unless a steel plate protective cover is used. Care shall be taken to make certain the use of a steel plate does not come in contact with the rails. See "Track Shield Details" for other requirements. Details of the shield shall be submitted to the Railroad for approval..

3) Wet the ties and other timber below the cutting area.

4) Monitor the work site for at least three hours after cutting for a smoldering fire.

B. Extensive overhead cutting will not be performed over the track area without the proper fire suppression equipment on-site and proper protection.

VII. UTILITIES

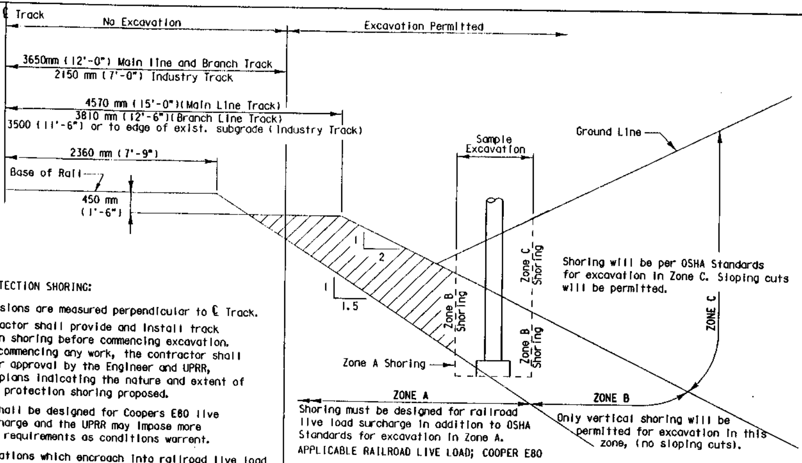
A. The demolition operations shall be planned such that the utility lines are operating safely at all times. The utility lines shall be protected if affected by demolition operations. All the work associated with utility lines should be coordinated by the contractor with the respective utility companies.

VIII. HAZARDOUS MATERIAL

A. If any hazardous materials are found, provide material protection as specified in local hazardous material codes and immediately contact the Railroad.

APPENDIX

- U.P.R.R. STANDARD DRAWING 106613
- TRACK SHIELD DETAIL
- FRAME PROTECTION DETAILS



TRACK PROTECTION SHORING:

All dimensions are measured perpendicular to ℄ Track.

The contractor shall provide and install track protection shoring before commencing excavation. Prior to commencing any work, the contractor shall submit for approval by the Engineer and UPRR, detailed plans indicating the nature and extent of the track protection shoring proposed.

Shoring shall be designed for Coopers E80 live load surcharge and the UPRR may impose more stringent requirements as conditions warrent.

For excavations which encroach into railroad live load surcharge zone, shoring plans will be accompanied by a copy of the design calculations, and both must be stamped by a registered professional engineer.

Design of shoring shall comply with UPRR guidlines for design and construction of shoring adjacent to active railroad tracks.

TRACK PROTECTION SHORING REQUIREMENTS

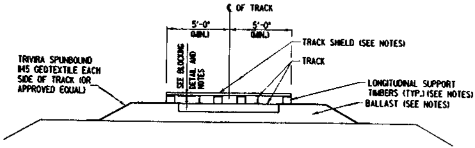
UNION PACIFIC RAILROAD

GENERAL SHORING REQUIREMENTS

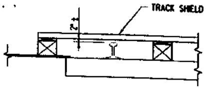
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98 REDRAWN

C.E. 106613



TRACK SHIELD DETAIL
FOR DEBRIS FALLING FROM BRIDGE DECK REMOVAL
(WHEN TRACK TIME WINDOW IS AVAILABLE)



BLOCKING DETAIL

NOTES:

1. A FLAGMAN IS REQUIRED AT ALL TIMES DURING THE USE OF A TRACK SHIELD.
2. THE TRACK SHIELD SHALL BE DESIGNED BY THE CONTRACTOR AND SHALL BE OF SUFFICIENT STRENGTH TO SUPPORT THE ANTICIPATED LOADS, INCLUDING IMPACT. THE SHIELD SHALL PREVENT ANY MATERIALS, EQUIPMENT OR DEBRIS FROM FALLING ONTO THE RAILROAD TRACK. ADDITIONAL LAYERS OF MATERIALS SHALL BE FURNISHED AS NECESSARY TO PREVENT FINE MATERIALS OR DEBRIS FROM SIFTING DOWN UPON THE TRACK.
3. THE SHIELD SHOULD PREFERABLY BE PREFABRICATED AND FURNISHED WITH LIFTING HOOKS TO SIMPLIFY REMOVAL.
4. THE SHIELD SHALL BE OF SUFFICIENT STRENGTH TO SPAN BETWEEN IT'S SUPPORTS WITHOUT BEARING UPON THE RAILS AND TO WITHSTAND DROPPING RUBBLE.
5. BEFORE REMOVAL, THE SHIELD SHALL BE CLEANED OF ALL DEBRIS AND FINE MATERIAL.
6. THE TRACK SHIELD SHALL EXTEND AT LEAST 20 FEET BEYOND THE LIMITS OF DEMOLITION TRANSVERSE TO THE EDGE OF THE BRIDGE.
7. LONGITUDINAL SUPPORT TIMBERS FOR THE SHIELD SHALL NOT EXTEND ABOVE THE TOP OF RAIL WHEN THE SHIELD IS REMOVED. BLOCKING FROM THE TOP OF RAIL TO THE BOTTOM OF THE SHIELD MAY BE ATTACHED TO THE SHIELD. REMAINING TIMBERS SHALL BE ANCHORED.
8. FOR TRAIN PASSAGE, THE RUBBLE SHALL BE REMOVED TO A MINIMUM OF 8'6" FROM THE NEAREST RAIL AND TO AN ELEVATION NO HIGHER THAN THE TOP OF RAIL.
9. AT THE END OF THE DAY, THE RUBBLE SHALL BE REMOVED COMPLETELY TO A MINIMUM OF 10'0" FROM THE NEAREST RAIL AND DOWN TO ORIGINAL GRADE.
10. CARE SHALL BE TAKEN TO NOT PLACE METAL ACROSS THE TRACK RAILS. RAILROAD COMMUNICATIONS ARE SENT THROUGH THE RAILS AND WILL BE DISRUPTED BY A SHORT BETWEEN RAILS.
11. DETAILS SHOWN APPLY FOR TIMBER TIES. SPECIAL DETAILS ARE REQUIRED FOR CONCRETE TIES.

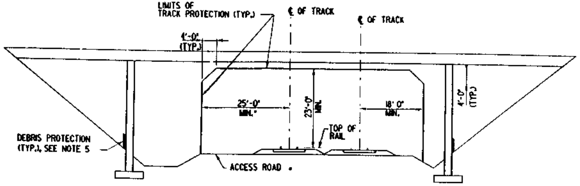
UNION PACIFIC RAILROAD

TRACK SHIELD DETAIL

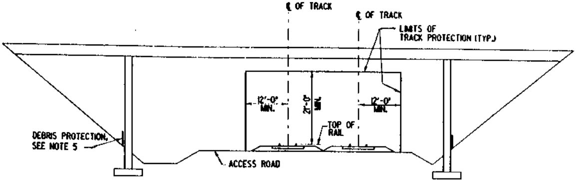
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 1 OF 1



BRIDGE ELEVATION
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION



BRIDGE ELEVATION
MINIMUM LIMITS OF PROTECTION FOR FRAME PROTECTION
(SPECIAL PERMISSION REQUIRED, SEE NOTE 1)

NOTES:

1. THE STANDARD LIMITS OF PROTECTION NOTED ARE THE MIN. CLEARANCES ALLOWED WITHOUT SPECIAL PERMISSION FROM THE RAILROAD. THE REDUCED CLEARANCES NOTED MAY BE ALLOWED BY THE RAILROAD. SPECIAL PERMISSION FOR THE REDUCED CLEARANCES IS REQUIRED FROM THE RAILROAD SERVICE UNIT SUPERINTENDENT.

2. THE PROTECTION FRAME SHALL AS A MINIMUM MATCH THE DEMOLITION LIMITS SHOWN AND EXTEND PAST THE BRIDGE WIDTH AS SHOWN ON THE ATTACHED DEMOLITION PLAN SHEET.

3. FOR ADDITIONAL CLEARANCE AND PROTECTION INFORMATION, SEE UNION PACIFIC RAILROAD STANDARD DRAWING NO. 0035

4. THE PROTECTION FRAME SHALL PREVENT DEMOLITION DEBRIS, DUST AND FINE MATERIAL FROM FALLING ONTO THE RAILROAD TRACKS, ACCESS ROAD OR TRAINS. THE FRAME SHALL BE DESIGNED BY THE CONTRACTOR TO SUPPORT THE ANTICIPATED DEMOLITION LOADS, AND IN ACCORDANCE WITH UNION PACIFIC GUIDELINES FOR DESIGN OF FALSEWORK FOR STRUCTURES OVER THE RAILROAD.

5. DEBRIS PROTECTION IS REQUIRED NEAR THE BASE OF THE SIDE SLOPES AND ADJACENT TO ROADS USED BY DEMOLITION EQUIPMENT TO PREVENT DEBRIS FROM ROLLING ONTO THE TRACK, ACCESS ROAD OR DITCH. USE TIMBERS AS REQUIRED TO STOP LARGE PIECES OF ROLLING DEBRIS.

6. ANY ACTIVITY WITHIN 25 FEET OF THE NEAREST RAIL OF A TRACK REQUIRES A FLAGMAN.

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE OF DETAIL.

UNION PACIFIC RAILROAD

FRAME PROTECTION DETAILS

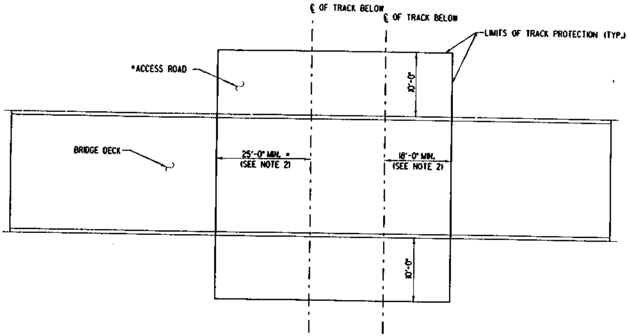
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

SHEET 1 OF 2

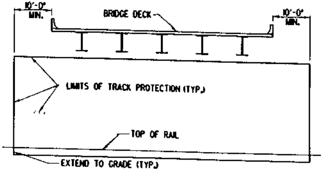
NOTES:

1. SEE GENERAL NOTES ON BRIDGE ELEVATION SHEET
2. STANDARD LIMITS OF PROTECTION ARE SHOWN. FOR MIN. LIMITS OF PROTECTION DIMENSIONS, SEE BRIDGE ELEVATION, MINIMUM LIMITS OF PROTECTION.



BRIDGE PLAN
STANDARD LIMITS OF PROTECTION FOR FRAME PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE



BRIDGE DECK CROSS SECTION
STANDARD LIMITS OF PROTECTION

* IF NO ACCESS ROAD, USE MIN. DIMENSION FROM OTHER SIDE

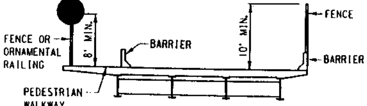
UNION PACIFIC RAILROAD

FRAME PROTECTION DETAILS

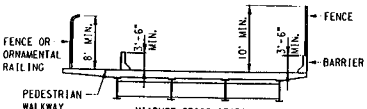
OFFICE OF CHIEF ENGINEER DESIGN

DATE: 3-31-98

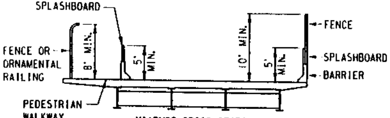
SHEET 2 OF 2



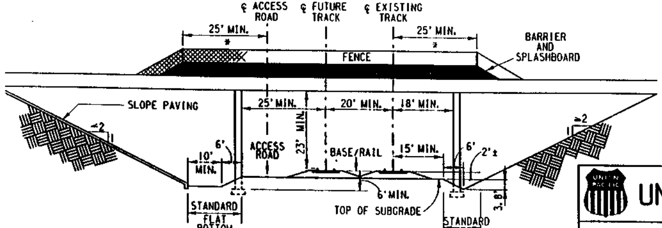
VIADUCT CROSS SECTION
NO SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH 3'-6" SOLID BARRIER AND FENCE
FOR SNOW REMOVAL AREAS



VIADUCT CROSS SECTION
WITH SPLASH BOARD AND FENCE
FOR SNOW REMOVAL AREAS



ELEVATION
PERPENDICULAR TO TRACKS

* Fences, splashboards, or solid barriers if required shall extend 25ft. beyond centerline of outer most track or access roadway.

GENERAL

Fence shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures.

Splashboards or solid 3'-6" high barrier rail shall be provided as indicated on the cross sections and elevation view on both sides of the viaduct in ALL new or modified structures where snow removal is being performed.

Lights are to be installed on the underside of the viaduct where shadows cast by the structure would interfere with Railroad operations.

Slope paving shall be provided where end slopes exceed 2 horizontal to 1 vertical.

Falsework for costruction of overhead structures shall comply to UPRR guidelines.

Demolition of existing overhead structures shall comply to UPRR guidlines.

Temporary shoring shall be designed in accordance with UPRR's Shoring Requirements (Drawing No. 106613) and UPRR guidlines.

Applicant shall be responsible for identification, location, and protection of existing utilities.

Contact UPRR's "Call Before You Dig" at least 48 hours prior to commencing work at 1-800-336-9193 to determine location of fiber optics.

Exceptions to these standards must be approved by UPRR's Chief Engineer Design.

CLEARANCES

Minimum vertical clearance shall be 23 feet above the plane of top-of-rails. Additional clearance may be required for construction purposes or if sag of vertical curve must be adjusted or if future track raise for flood considerations or maintenance is probable.

Minimum horizontal clearances, measured at right angle from centerline of track, shall be as shown in elevation view.

Minimum construction clearances shall be 21 feet vertical above the plane of top-of-rails and 12 feet horizontal at right angle from centerline of track.

FUTURE TRACKS

Space is to be provided for one or more future tracks as required for long range planning or other operating requirements. Where provision is made for more than two tracks, space is to be provided for access road on both sides of tracks.

PIERS

Pier protection (crash walls) shall be provided in accordance with AREA Chapter 8, Part 2.1.5 for piers within 25 feet of the centerline of track.

Top of footings within 25' from centerline of track shall be a minimum of 6 feet below base of rail and a minimum of 1 foot below flow line of ditch.

DRAINAGE

Drainage from the overpass shall be diverted away from UPRR's tracks and not discharged onto the tracks or roadbed.

A standard "V"-shaped or flat-bottom ditch shall be provided on each side of the tracks as necessary.

Culverts may be installed on opposite side of column from track in lieu of standard Railroad ditches when approved by Chief Engineer Design. Maintenance of culverts is to be at applicant's expense.

UNION PACIFIC RAILROAD

BARRIERS AND CLEARANCES TO BE PROVIDED AT HIGHWAY, STREET, AND PEDESTRIAN OVERPASSES

OFFICE OF CHIEF ENGINEER DESIGN

REVISED: MAR. 31, 1998 STD DWG 0035

BRIDGE CONSTRUCTION MEMO 125-0.0

FORMS

October 19, 1998

Sheet 1 of 1

Volume II

TABLE OF CONTENTS FOR SECTION NO. 125

Memo No.	Date	Title
125-1.0	10-19-98	LOST DECK FORMS
125-2.0	07-01-75	SOFFIT FORMS

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

Volume II

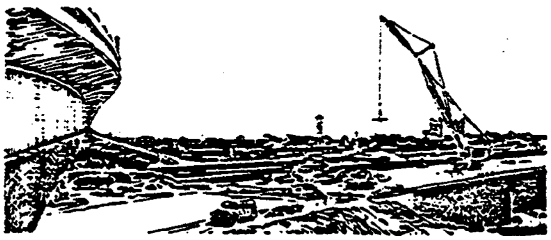
LOST DECK FORMS

Supporting Lost Deck Forms

It is permissible to cast pieces of reinforcing steel into the interior faces of box girder stems to support “lost deck” forms. When these pieces of reinforcing are used, they should be no larger than a #19, 19.1 mm ∅ (#6, .75” ∅) bar and should be at least 25 mm (1”) clear from any permanent reinforcing.

Patching Lost Deck Forms

Paper is not to be used to patch cracks or holes in “lost deck” forms. Metal or wood is acceptable provided it does not infringe on the required deck thickness.



BRIDGE CONSTRUCTION MEMO 125-2.0

FORMS

July 1, 1975

Volume II

SOFFIT FORMS

Box girder soffit forms should be constructed so that each end joint is fully supported by a joist. This permits nailing the ends of plywood sheets, which tends to eliminate offsets in the concrete at the joints.

BRIDGE CONSTRUCTION BULLETIN

Approved:____Original signed by_________
DOLORES VALLS, Deputy Division Chief
Offices of Structure Construction

File: BCM 125-3.0
FORMS

Date: December 12, 2001
Expires: December 12, 2002
Supersedes: None

Subject: Pre-fabricated Soffit Panels

Prefabricated soffit forming panels (gang forms) are being used more commonly in conventional falsework systems. These prefabricated soffit forming panels typically consist of an eight-foot (8 ft.) wide and up to 40 ft. long panel comprised of plywood nailed to 2x8 joist (see Figure No. 1). The use of prefabricated soffit panels have proven to be a reliable, efficient, falsework soffit forming system.

When the soffit gang form panels are erected onto the falsework stringers they are typically placed with a gap between each panel. This gap aids in the erection and removal of the panel system. This gap is bridged with a filler strip of plywood (see Figure No. 2). Typically, these filler strips are 12" wide.

The filler strip between panels currently violates Section 51-1.05 of the Standard Specifications. This section requires a minimum forming panel dimension of 3' x 6' (0.9m x 1.8m).

Recognizing the construction benefits of the prefabricated soffit panel system the Offices of Structure Construction has determined that a uniform one-foot wide minimum filler strip is acceptable when a prefabricated soffit panel forming system is used. Figure No. 3 depicts the final product.

If a contractor desires to use prefabricated falsework soffit panels, the RE/Structure Representative shall accommodate the request and issue a no-cost/credit contract change order. See attachment No. 1 for an example CCO and Memorandum.

As with any forming system, poor workmanship and materials can lead to undesirable results. Common issues with this type of forming system are:

- Loose filler strips (i.e. not firmly nailed to the joist)
- Damaged and worn panels from overuse.
- New replacement plywood forming sheets next to seasoned sheets.
- New plywood filler strips adjacent to seasoned panel sheets.
- Non-uniform filler strip widths.
- Non-uniform form line patterns. Skewed bridges exacerbate this situation.

All of the above issues can be successfully mitigated with the timely enforcement of Section 51-1.05 of the Standard Specifications. Frequently, repaired portions of the gang forms, or the filler strips, are replaced with new plywood forming material. New plywood next to old plywood will produce an unacceptable non-uniform concrete surface. One method proven successful to age the new forming material is to apply a cement and water paste, allow the paste to dry, and then remove. The dried cement paste absorbs the fresh wood sugar from the

c: BCR&P Manual Holders
CMB - Consultant Firms
RPieplow, HQ Division of Construction

new plywood and ages the wood so that the finished concrete will have a color and texture similar to the color and texture of the seasoned plywood forms.

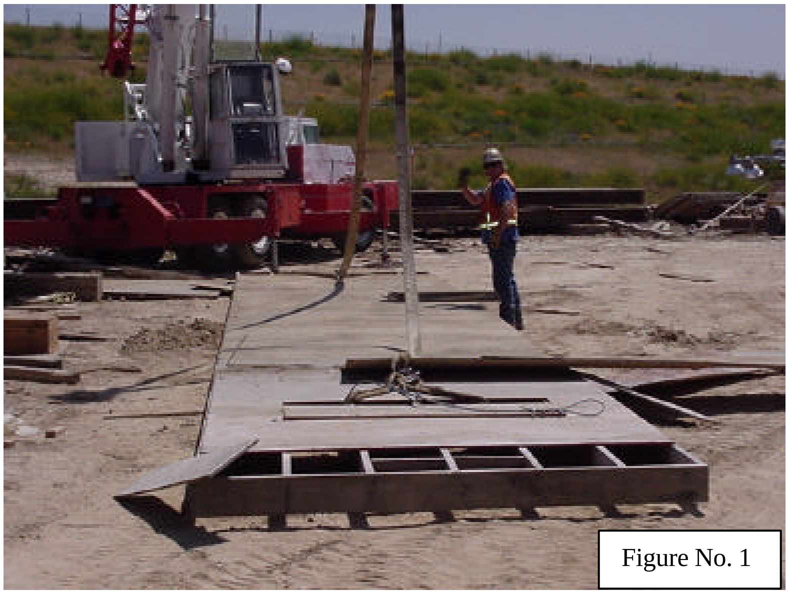


Figure No. 1

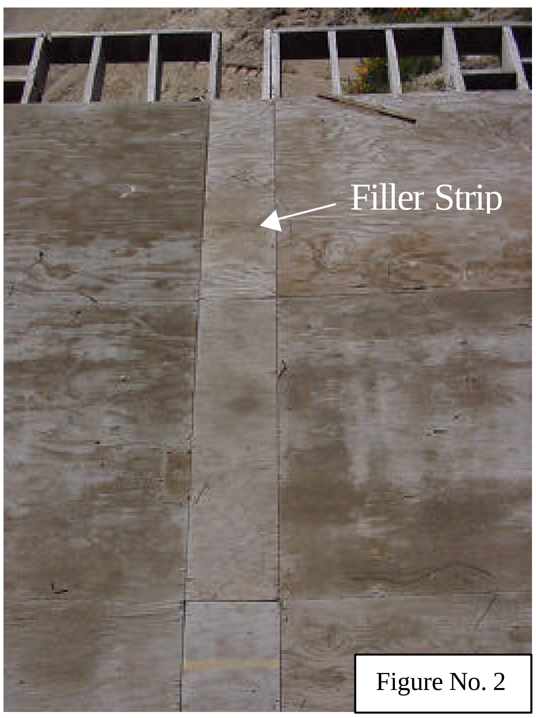


Figure No. 2

Figure No. 3

c: BCR&P Manual Holders
Consultant Firms
RPieplow, HQ Division of Construction

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

Sheet 1 of

Contract Change Order

CEM-4900 (OLD HC-5 REV. 8/97) CT # 7541-3501-0

Change Requested by: ☐ **Engineer** X **Contractor**

CCO No.	Suppl. No.	Contract No.	Road	Federal Number(s)

To ***Contractor***

You are directed to make the following changes from the plans and specifications or do the following described work not included in the plans and specifications for this contract. ***NOTE: This change order is not effective until approved by the Engineer.***

escription of work to be done, estimate of quantities and prices to be paid. (Segregate between additional work at contract price, agreed rice and force account.) Unless otherwise stated, rates for rental of equipment cover only such time as equipment is actually used and no llowance will be made for idle time. The last percentage shown is the net accumulated increase or decrease from the original quantity in the ngineer's Estimate

NO COST, CHANGE IN SPECIFICATIONS:

Insert the following after the last sentence of the 11th paragraph of Section 51-1.05 FORMS of the Standard Specifications.

When prefabricated soffit panels are utilized, form filler panels joining the prefabricated panels shall be of uniform length and have a uniform minimum width of 0.3m and shall produce a smooth uniform surface with consistent longitudinal and transverse lines between the prefabricated panels.

There shall be no cost or credit to the State by reason of this change.

Estimated Cost: Decrease ☐ **Increase** ☐ **$**

By reason of this order the time of completion will be adjusted as follows:

ubmitted by

Signature	(Print name & title)	Date

pproval Recommended by

Signature	(Print name & title)	Date

ngineer Approval by

Signature	(Print name & title)	Date

We the undersigned contractor, have given careful consideration to the change proposed and agree, if this proposal is approved, that we will provide all equipment, furnish the materials, except as may otherwise be noted above, and perform all services necessary for the work above specified, and will accept as full payment therefor the prices shown above. **NOTE: If you, the contractor, do not sign acceptance of this order, your attention is directed to the requirements of the specification as to proceeding with the ordered work and filing a written protest within the time therein specified.**

ontractor Acceptance by

Signature	(Print name & title)

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

CONTRACT CHANGE ORDER MEMORANDUM

DC-CEM-4903 (OLD HC-39 REV. 6/93) CT# 7541-3544-0

DATE	
TO	FILE EA NUMBER CO -- RTE -- PM FEDERAL NUMBER
FROM	
CCO NO. / SUPPLEMENT NO. / CATEGORY CODE	CONTINGENCY BALANCE (including this change)
$ INCREASE ☐ DECREASE ☐	HEADQUARTERS APPROVAL REQUIRED? YES ☐ NO ☐
SUPPLEMENTAL FUNDS PROVIDED $	IS THIS REQUEST IN ACCORDANCE WITH ENVIRONMENTAL DOCUMENTS? YES ☐ NO ☐

THIS CHANGE ORDER PROVIDES FOR:

Modifying Section 51-1.05 FORMS of the Standard Specifications to allow the use of prefabricated soffit panels.

REASON FOR CHANGE:

The Offices of Structure Construction recognizes the efficiency and safety benefits of falsework construction utilizing prefabricated soffit panels. Typical construction practice entails setting the prefabricated soffit panels onto the falsework stringers leaving a small 0.3 meter gap between each panel. This gap is then filled with a strip of plywood forming material.

Current specifications require form panels for exposed surfaces to have a minimum dimension of 0.9m by 1.8m. This change allows for a unifom, minimum 0.3m wide filler strip when a prefrabricated soffit panel system is used.

METHOD OF PAYMENT: No cost change

TIME ADUSTMENT: There will be no time adjustment for this change.

CONCURED BY:	DATE	ESTIMATE OF COST	THIS REQUEST	TOTAL TO DATE
CONSTRUCTION ENGINEER	DATE	ITEMS		
BRIDGE ENGINEER	DATE	FORCE ACCOUNT		
FHWA REPRESENTATIVE	DATE	AGREED PRICE		
		ADJUSTMENT		
PROJECT ENGINEER	DATE	**TOTAL**		
OTHER (SPECIFIY)	DATE	**FEDERAL PARTICIPATION** ☐ PARTICIPATING ☐ PARTICIPATING IN PART ☐ NONE ☐ NON-PARTICIPATING (MAINTENANCE) ☐ NON-PRATICIPATING		
	DATE	FEDERAL SEGREGATION (IF MORE THAN ONE FUNDING SOURCE OR P.I.P. TYPE) ☐ CCO FUNDED PER CONTRACT ☐ CCO FUNDED AS FOLLOWS		
DISTRICT PRIOR APPROVAL BY	DATE	FEDERAL FUNDING SOURCE	PERCENT	
HQ (ISSUE & APPROVE) (TO PROCEED) BY	DATE	_____	_____	
		_____	_____	
RESIDENT ENGINEER SIGNATURE	DATE	_____	_____	
		_____	_____	

HC-39 Word(Rev.9/96)

BRIDGE CONSTRUCTION MEMO 130-0.0

FOUNDATIONS

August 27, 2003

Volume II

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130-1	10/15/2002	FOUNDATION TESTING BRANCH
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130-3.0	10/19/1981	SETTLEMENT PLATFORMS
130-4.0	11/9/1981	SEAL COURSES
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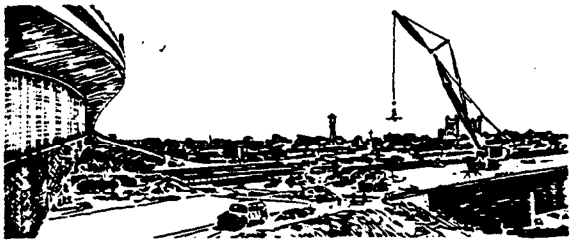
130-9.0	10/19/2001	MITIGATION OF CIDH PILES CONSTRUCTION USING THE SLURRY DISPLACEMENT METHOD
130-10	7/1/1999	CIDH PILE INFORMATION SUBMITTAL
130-11	7/1/1999	SLURRY TEST KITS FOR CIDH PILES
130-12	1/14/2000	APPROVED SYNTHETIC DRILLING SLURRIES
130-13	8/27/03	PILE DRIVING ACCEPTANCE CRITERIA

Jeff Abercrombie

JEFF J. ABERCROMBIE, Acting Deputy Division Chief
Offices of Structure Construction



STANDARD PILE LOAD TEST PROCEDURES

Reference: Section 49-1.10, Load Testing, of the Standard Specifications

The Standard Specifications require that, "When load test piles are shown on the plans of a structure, the loading tests of said piles shall be completed before the remaining piles for that structure are cast, cut to length, or driven, unless other limitations are specified in the special provisions." When the special provisions refer to specified control areas, it will be necessary to load test piles within the specified control area before any other piles can be cast, cut to length, or driven within the control area.

Load test piles are to be driven only to the specified tip elevation. Do not drive the test pile below specified tip for the purpose of obtaining the specified bearing. However, if the ENR equation indicates that the test pile has an extremely low bearing value (less than 50% of the required bearing value), the Structure Representative should contact the Engineering Geology Branch to determine whether or not the test pile should be driven to a lower tip elevation. Geology should be contacted as soon as possible as this determination should preferably be made before the Contractor moves his pile driving rig from the test pile. The anchor piles are to be driven so as to have both specified tip and specified bearing.

Pile load tests will normally be conducted by the Engineering Geology Branch. Note that the Engineering Geology Branch should be given as much advance notice as possible regarding the date that load testing will be required. It is necessary that the Contractor assist the Geology Branch with the pile load testing. In order to compensate the: Contractor for this assistance, it is necessary that a Contract Change Order be prepared by the Structure Representative in advance of starting the pile load tests. Any questions concerning the location, driving, or preparation of test piles, as well as anchor piles, should be referred to the Engineering Geology Branch.

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 130-1 FOUNDATIONS**
Approved: Original Signed by Dolores Valls **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 130-1 Dated December 2, 1996**

Subject: Foundation Testing Branch

The Foundation Testing Branch, Office of Geotechnical Support, Geotechnical Services, provides statewide foundation testing services for structure construction projects. These services include:

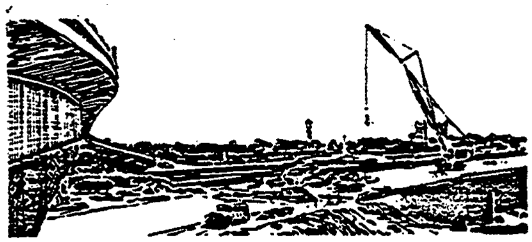
- Static load tests for compression, tension, and lateral capacities of piles
- Pile Dynamic Analyses
- Nondestructive testing of cast-in-drill-hole (CIDH) piles (Gamma-Gamma and Crosshole Sonic Logging)
- CIDH Pile Mitigation Review
- Pile drivability studies
- Field acceptance criteria by wave equation analysis
- Sonic Caliper Testing
- Vibration Monitoring
- Geotechnical Testing Consultation

For contracts that require any of the listed testing services as required by the contract documents, contact Foundation Testing in Sacramento. For the appropriate contact use the contact link on the following web page:

http://www.dot.ca.gov/hq/esc/geotech/index.htm

Test request forms can be found at this same web address under the 'Requests' link. Request forms also have contact information as well as a fax number for form transmittal. Please allow 3 weeks advance notice for these services.

c: BCR&P Manual Holders
Consultant Firms
R. Pieplow, HQ Const.
J. Van Velsor, GS



BRIDGE CONSTRUCTION MEMO 130-2.0

FOUNDATIONS

January 11, 1988

Volume II

PILE HAMMER DATA

The attached charts present pile hammer data for pile hammers currently in use in construction.

In the determination of the safe bearing values of piles, Section 49-1-08 of the Standard Specifications requires the use of the manufacturers' ratings of the energies of pile hammers. The attached charts list these energy ratings as well as other hammer properties published by the manufacturers.

Structure Representatives are cautioned that, even under ideal hammer operating conditions, energy dissipation inherent in the impact as well as in the hammer mechanism may greatly reduce the energy delivered to the pile and may give erroneous safe bearing values. Improper or inadequate hammer operation will, of course, compound the energy losses. Additionally, the energy of some hammers can be varied by changing fuel settings, using interchangeable rams, etc. If the Structure Representative is not certain of the energy rating of the hammer being used, the Contractor should be required to furnish Manufacturer's data for the hammer.

In order that the pile hammer data may be kept current, please furnish information to the Office of Structure Construction of any omissions of new or old hammers, or any errors in the hammer properties.

5/F

AIR/STEAM HAMMERS

RATED ENGERGY (FT.-LBS.)	MODEL	MANUFACTURERS	TYPE	STYLE	BLOWS PER (MIN.)	WT. OF STRIKING PARTS (LBS.)	TOTAL WEIGHT (LBS.)	HAMMER LENGTH (FT.-IN.)	JAW DIMENSIONS	INLET PRESSURE (PSI)
1,800,000	6300	VULCAN	SGL.-ACT	OPEN	42	300,000	575,000	30'0"	22"x144"	235
1,582,220	MRBS 12500	MENCK	SGL.-ACT	OPEN	36	275,580	540,130	35'9"	CAGE	171
1,200,000	2000E6	CONMACO	SGL.-ACT.	OPEN	40	200,000	490,000	35'6"	CAGE	155
1,050,000	1750E6	CONMACO	SGL.-ACT.	OPEN	40	175,000	465,000	35'6"	CAGE	135
867,960	MRBS 8000	MENCK	SGL.-ACT.	OPEN	38	176,370	330,690	30'10"	CAGE	171
750,000	1500E5	CONMACO	SGL.-ACT.	OPEN	42	150,000	283,000	30'6"	14 1/2"x120"	135
750,000	5150	VULCAN	SGL.-ACT.	OPEN	46	150,000	275,000	26'3 1/2"	22"x120"	175
510,000	850E6	CONMACO	SGL.-ACT.	OPEN	40	85,000	173,600	25'2"	18 3/4"x100"	160
500,000	5100	VULCAN	SGL.-ACT.	OPEN	48	100,000	197,000	27'4"	22"x120"	150
499,070	MRBS 4600	MENCK	SGL.-ACT.	OPEN	42	101,410	176,370	27'5"	CAGE	142
350,000	700E5	CONMACO	SGL.-ACT.	OPEN	43	70,000	152,000	23'2'	18 3/4"x100	130
325,480	MRBS 3000	MENCK	SGL.-ACT.	OPEN	42	66,135	108,025	25'0"	CAGE	142
300,000	3100	VULCAN	SGL.-ACT.	OPEN	60	100,000	195,500	23'3"	18 3/4"x88"(M)	130
300,000	560	VULCAN	SGL.-ACT.	OPEN	47	62,500	134,060	23'0'	18 3/4"x88"(M)	150
225,000	450E5	CONMACO	SGL.-ACT.	OPEN	45	45,000	103,000	23'-3"	14"x80"	130
200,000	540	VULCAN	SGL.-ACT.	OPEN	48	40,900	102,980	22'7"	14"80"(M)	130
189,850	MRBS 1800	MENCK	SGL.-ACT.	OPEN	44	38,580	64,590	22'5"	CAGE	142
180,000	360	VULCAN	SGL.-ACT.	OPEN	62	60,000	124,830	19'0"	18 3/4"x88"(M)	130
150,000	300E5	CONMACO	SGL.-ACT	OPEN	40	30,000	58,400	20'10"	11 1/4"x56"	135
150,000	530	VULCAN	SGL.-ACT.	OPEN	42	30,000	57,680	20'5"	10 1/2"x54"	150
150,000	60X	RAYMOND	SGL.-ACT.	OPEN	60	60,000	85,000	22'7"	——	165
120,000	340	VULCAN	SGL.-ACT.	OPEN	60	40,000	98,180	18'7"	14"x80"	120
100,000	200E5	CONMACO	SGL.-ACT.	OPEN	46	20,000	48,000	19'1"	11 1/4"x56"	110
100,000	40X	RAYMOND	SGL.-ACT.	OPEN	64	40,000	62,000	19'1"	——	135
93,340	MRBS 850	MENCK	SGL.-ACT.	OPEN	45	18,960	27,890	19'8"	CAGE	142
90,000	030	VULCAN	SGL.-ACT.	OPEN	54	30,000	53,470	16'4"	11 1/4"x56"	150
90,000	300E3	CONMACO	SGL.-ACT.	OPEN	52	30,000	55,390	16'10"	11 1/4"x56"	150
81,250	8 0	RAYMOND	SGL.-ACT.	OPEN	40	25,000	34,000	19'4"	10 1/4"x25"	135
75,000	30X	RAYMOND	SGL.-ACT.	OPEN	70	30,000	52,000	19'1"	——	150
62,500	125E5	CONMACO	SGL.-ACT.	OPEN	41	12,500	22,000	18'0"	9 1/4"x26"	100
60,000	200E3	CONMACO	SGL.-ACT.	OPEN	55	20,000	44,560	15'0"	11 1/4"x56"	110
60,000	S-20	MKT	SGL.-ACT.	CLOSED	60	20,000	38,650	15'5"	—x36"	150
60,000	020	VULCAN	SGL.-ACT.	OPEN	59	20,000	41,670	14'8"	11 1/4"x37"	120
57,500	115E5	CONMACO	SGL.-ACT.	OPEN	42	11,500	21,000	17'9"	9 1/4"x26"	100
56,875	5/0	RAYMOND	SGL.-ACT.	OPEN	44	17,500	26,450	16'9"	10 1/4"x25"	150
50,000	100E5	CONMACO	SGL.-ACT.	OPEN	47	10,000	19,500	17'9"	9 1/4"x26"	100
48,750	160E3	CONMACO	SGL.-ACT.	OPEN	50	16,250	33,200	13'10"	11 1/4"x42"	100
50,000	200-C	VULCAN	DIFFER.	OPEN	95	20,000	39,000	13'11"	11 1/4"x37"	142
48,750	016	VULCAN	SGL.-ACT.	OPEN	58	16,250	30,250	13'11"	11 1/4"x32"	120
48,750	4·0	RAYMOND	SGL.-ACT.	OPEN	46	15,000	23,800	16'1"	——	120
48,750	150-C	RAYMOND	DIFFER.	OPEN	95-105	15,000	32,500	15'9"	——	120
45,200	MRBS 500	MENCK	SGL.-ACT.	OPEN	48	11,020	15,210	16'8"	—x26"	142
44,000	MS-500	MKT	SGL.-ACT.	OPEN	40-50	11,000	17,000	16'8"	—x26"	115
42,000	140E3	CONMACO	SGL.-ACT.	OPEN	55	14,000	30,750	13'10"	11 1/4"x42"	100
42,000	014	VULCAN	SGL.-ACT.	OPEN	59	14,000	27,500	13'8"	11 1/4"x32"	110
40,600	3 0	RAYMOND	SGL.-ACT.	OPEN	50	12,500	21,000	15'7"	10 1/4"x25"	120
40,000	80E5	CONMACO	SGL.-ACT.	OPEN	47	8,000	17,500	17'9"	9 1/4"x26"	80

RATED ENGERGY (FT.-LBS.)	MODEL	MANUFACTURERS	TYPE	STYLE	BLOWS PER (MIN.)	WT. OF STRIKING PARTS (LBS.)	TOTAL WEIGHT (LBS.)	HAMMER LENGTH (FT.-IN.)	JAW DIMENSIONS	INLET PRESSURE (PSI)
37,500	S-14	MKT	SGL.-ACT.	CLOSED	60	14,000	31,700	13'7"	x36"	100
37,375	115E3	CONMACO	SGL.-ACT.	OPEN	52	11,500	20,830	14'2"	91/4"x32"	100
36,000	140-C	VULCAN	DIFFER.	OPEN	101	14,000	27,984	12'3"	111/4"x32"	140
32,885	100-C	VULCAN	DIFFER.	OPEN	103	10,000	22,200	14'0"	91/4"x26"	140
32,500	100E3	CONMACO	SGL.-ACT.	OPEN	55	10,000	19,280	14'2"	91/4"x32"	100
32,500	65E5	CONMACO	SGL.-ACT.	OPEN	80	6,500	12,500	16'10"	81/4"x20"	95
32,500	2/0	RAYMOND	SGL.-ACT.	OPEN	50	10,000	18,550	15'0"	101/4"x25"	110
32,500	010	VULCAN	SGL.-ACT.	OPEN	50	10,000	18,780	15'0"	91/4"x26"	105
32,500	S-10	MKT	SGL.-ACT.	CLOSED	55	10,000	22,380	14'1"	—x30"	80
30,800	MS-350	MKT	SGL.-ACT.	OPEN	40-50	7,716	11,500	16'2"	26	105
26,000	80E3	CONMACO	SGL.-ACT.	OPEN	56	8,000	17,280	14'2"	91/4"x32"	80
26,000	85-C	VULCAN	DIFFER.	OPEN	111	8,525	19,020	12'7"	91/4"x26"	128
26,000	08	VULCAN	SGL.-ACT.	OPEN	50	8,000	16,750	14'10"	91/4"x26"	83
26,000	S-8	MKT	SGL.-ACT.	CLOSED	55	8,000	18,300	14'4"	—x26"	80
25,000	50E5	CONMACO	SGL.-ACT.	OPEN	48	5,000	11,000	16'10"	81/4"x20"	70
24,450	80-C	VULCAN	DIFFER.	OPEN	109	8,000	17,885	12'7"	91/4"x26"	120
24,450	80-C(HYD)	RAYMOND	DIFFER.	OPEN	110-120	8,000	17,780	11'10"	——	5,100
24,450	80-C	RAYMOND	DIFFER.	OPEN	95-105	8,000	17,885	12'2"	——	120
24,375	0	RAYMOND	SGL.-ACT.	OPEN	50	7,500	16,000	15'0"	101/4"x25"	110
24,375	0	VULCAN	SGL.-ACT.	OPEN	50	7,500	16,250	15'0"	91/4"x26"	80
24,000	C-826	MKT	COMPOUND	CLOSED	85-95	8,000	17,750	12'2"	—x26"	125
19,500	65E3	CONMACO	SGL.-ACT.	OPEN	61	6,500	12,100	12'10"	91/4"x26"	100
19,500	65-C	RAYMOND	DIFFER.	OPEN	110	6,500	14,675	11'8"	91/4"x19"	120
19,500	1-S	RAYMOND	SGL.-ACT.	OPEN	58	6,500	12,500	12'9"	71/2"x281/4"	100
19,500	06(106)	VULCAN	SGL.-ACT.	OPEN	60	6,500	11,200	13'0"	81/4"x20"	100
19,500	65-C(HYD)	RAYMOND	DIFFER.	OPEN	130	6,500	14,615	12'1"	——	5,000
19,200	65-C	VULCAN	DIFFER.	OPEN	117	6,500	14,886	12'1"	81/4"x20"	150
19,150	11B3	MKT	DBL.-ACT.	CLOSED	95	5,000	14,000	11'2"	—x26"	100
19,150	1100	BSP	DBL.-ACT.	CLOSED	95	5,000	14,000	11'2"	—x26"	90
16,250	S-5	MKT	SGL.-ACT.	CLOSED	60	5,000	12,460	13'3"	—24"	80
16,000	C-5(STM)	MKT	DBL-ACT.	CLOSED	100-110	5,000	11,880	——	—x26"	100
15,100	50-C	VULCAN	DIFFER.	OPEN	117	5,000	11,782	11'0"	81/4"x20"	120
15,000	50E3	CONMACO	SGL.-ACT.	OPEN	64	5,000	10,600	12'10"	91/4"x26"	80
15,000	1(106)	VULCAN	SGL.-ACT.	OPEN	60	5,000	9,700	——	81/4"x20"	80
15,000	1	RAYMOND	SGL.-ACT.	OPEN	60	5,000	11,000	12'9"	71/2"x281/4"	80
14,200	C-5(AIR)	MKT	COMPOUND	CLOSED	100-110	5,000	11,880	8'9"	—x26"	100
13,100	10B3	MKT	DBL.-ACT.	CLOSED	105	3,000	10,850	9'2"	—x24"	100
13,100	1000	BSP	DBL.-ACT.	CLOSED	105	3,000	10,850	9'2"	—x24"	90
8,750	900	BSP	DBL.-ACT.	CLOSED	145	1,600	7,100	8'2"	—x20"	90
8,750	9B3	MKT	DBL.-ACT.	CLOSED	145	1,600	7,000	8'4"	81/2"x20"	100
7,260	30-C	VULCAN	DIFFER.	OPEN	133	3,000	7,036	8'11"	71/4"x19"	120
7,260	2	VULCAN	SGL.-ACT.	OPEN	70	3,000	6,700	11'7"	71/4"x19"	80
4,700	700N	BSP	DBL.-ACT.	CLOSED	225	850	6,500	5'5"	—x15"	90
4,150	7	MKT	DBL.-ACT.	CLOSED	225	800	5,000	6'1"	—x21"	100
4,000	DGH-900	VULCAN	DIFFER.	CLOSED	328	900	5,000	6'9"	VARIES	78
3,000	600N	BSP	DBL.-ACT.	CLOSED	250	500	3,800	5'0"	—x15"	90
2,500	6	MKT	DBL.-ACT.	CLOSED	275	400	2,900	5'3"	—x15"	100
1,200	500N	BSP	DBL.-ACT.	CLOSED	330	200	2,000	3'11"	—x12"	90
1,000	5	MKT	DBL.-ACT.	CLOSED	300	200	1,500	4'7"	6"x11"	100
386	DGH-100D	VULCAN	DIFFER.	CLOSED	303	100	786	4'2"	41/4"x83/4"	60

DIESEL HAMMERS

ENERGY RANGE (FT LB)	MODEL	MANUFACTURER	SINGLE/ DOUBLE ACTING	BLOWS PER MIN	PISTON WEIGHT LBS	TOTAL WEIGHT LBS	TOTAL LENGTH FT-IN	MAXIMUM STROKE FT-IN	WIDTH BETWEEN JAWS (IN)
300,000-138,000	D100	DELMAG	— —	33-50	23,600	43,200	20'4"	— —	— —
280,000-	K150	KOBE	SINGLE	45-60	33,100	80,500	29'8"	8'6"	CAGE
225,000-90,000	D-80-12	DELMAG	— —	35-50	19,500	35,900	20'4"	11'6"	— —
165,000-82,600	D62-22	DELMAG	SINGLE	34-50	14,600	27,900	17'9"	12'8"	32
149,600-88,000	MH80B	MITSUBISHI	SINGLE	42-60	17,600	43,600	19'6"	8'6"	— —
141,000-63,360	MB70	MITSUBISHI	SINGLE	38-60	15,840	46,000	19'6"	8'6"	— —
135,200-79,500	MH72B	MITSUBISHI	SINGLE	38-60	15,900	44,000	19'6"	8'6"	— —
127,500-90,000	DE150	MKT	SINGLE	40-50	15,000	32,150	19'10½"	11'8"	32
117,000-62,500	D55	DELMAG	SINGLE	36-47	12,100	26,300	17'9"	9'10"	32
105,600-	K60	KOBE	SINGLE	42-60	13,200	37,500	24'3"	8'0"	42
105,000-48,400	D46-23	DELMAG	SINGLE	37-53	10,120	19,900	17'3"	10'8"	32
93,500-66,000	DE-110	MKT	SINGLE	40-50	11,000	24,550	16'2"	11'8"	32
92,752	KC45	KOBE	SINGLE	39-60	9,920	24,700	17'11"	9'4"	— —
91,100	K45	KOBE	SINGLE	39-60	9,900	25,600	18'6"	9'2"	36
87,000-43,500	D44	DELMAG	SINGLE	37-56	9,500	22,300	15'10"	9'2"	32
84,300-50,250	MH45	MITSUBISHI	SINGLE	42-60	10,500	24,600	17'11"	8'6"	36
84,000-37,840	M43	MITSUBISHI	SINGLE	40-60	9,460	22,660	16'3"	8'10"	37
83,100-38,000	D36-23	DELMAG	SINGLE	37-53	7,900	17,700	14'11"	9'3"	32
79,500-	J44	IHI	SINGLE	42-70	9,720	21,500	14'10"	8'2"	37
79,000	B-500	BERMINGHAMMER	SINGLE	33-55	6,900	18,500	17'10"	13'0"	— —
79,000-	K42	KOBE	SINGLE	40-60	9,260	24,000	17'8"	8'6"	36
78,800-	B45	BSP	DOUBLE	80-100	10,000	27,500	19'3"	— —	36
73,000-40,150	3,400	F.E.C.	— —	40-60	7,500	14,600	16'0"	— —	26
72,182	KC35	KOBE	SINGLE	39-60	7,720	17,400	16'10"	9'4"	— —
70,800	K35	KOBE	SINGLE	39-60	7,700	18,700	17'8"	9'2"	30
70,000-36,100	1070	ICE	DOUBLE	64-68	10,000	21,500	17'10"	— —	30
66,100-33,700	D30-23	DELMAG	— —	38-54	6,600	13,150	20'5"	— —	— —
65,600-38,600	MH35	MITSUBISHI	SINGLE	42-60	7,720	18,500	17'3"	8'6"	32
64,000-29,040	M33	MITSUBISHI	SINGLE	40-60	7,260	16,940	13'2	8'0"	32
63,900-	B35	BSP	DOUBLE	80-100	7,700	21,200	18'5"		36
63,500-	J35	IHI	SINGLE	72-70	7,730	16,900	14'6"	8'3"	32
63,000-34,650	3000	F.E.C.	SINGLE	40-60	6,600	13,200	15'6"	10'6"	26
60,100	K32	KOBE	SINGLE	40-60	7,050	17,750	17'8"	8'6"	30
59,500-42,000	DE70/50B	MKT	SINGLE	40-50	7,000	14,700	16'11"	11'9"	26
54,250-23,800	D30	DELMAG	— —	39-60	6,600	12,300	— —	— —	— —
51,518	KC25	KOBE	SINGLE	39-60	5,510	12,130	16'10"	9'4"	— —
50,700	K25	KOBE	SINGLE	39-60	5,510	13,100	17'6"	9'3"	26
50,000-27,500	2,500	F.E.C.	— —	40-60	2,750	6,540	14'0"	— —	20
50,000-25,400	660	ICE	DOUBLE	84-88	7,564	24,480	17'4"	— —	30
48,500-24,500	D22-23	DELMAG	SINGLE	38-54	4,850	11,400	17'2"	10'7"	26
46,900-27,550	MH25	MITSUBISHI	SINGLE	42-60	5,510	13,200	16'8"	8'6"	28
46,000	B-400	BERMINGHAMMER	SINGLE	44-60	5,000	12,400	14'10"	11'0"	— —
45,700	B25	BSP	DOUBLE	80-100	5,510	15,200	17'9"	— —	30
45,000-20,240	M23	MITSUBISHI	SINGLE	42-60	5,060	11,220	14'1"	8'10"	26
42,500-30,000	DA-55C	MKT	DOUBLE	40-50	5,000	17,000	17'0"	11'10"	28
42,500-30,000	DE70/50B	MKT	SINGLE	40-50	5,000	12,700	16'11"	11'9"	26
41,300	K22	KOBE	SINGLE	40-60	4,850	12,350	17'6"	9'2"	26
40,000-25,400	640	ICE	DOUBLE	74-77	6,000	15,600	16'2"	— —	26
39,700	D22	DELMAG	SINGLE	42-60	4,850	11,400	14'2"	8'2"	26
39,100	J22	IHI	SINGLE	42-70	4,850	10,800	14'0"	10'0"	26
34,000	B-300	BERMINGHAMMER	SINGLE	40-60	3,750	9,625	14'0"	10'9"	— —
34,000-24,000	DA-45	MKT	SINGLE	40-50	4,000	14,200	15'1"	10'9"	28

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DIESEL HAMMERS

ENERGY RANGE (FT LB)	MODEL	MANUFACTURER	SINGLE/ DOUBLE ACTING	BLOWS PER MIN.	PISTON WEIGHT LBS	TOTAL WEIGHT LBS	TOTAL LENGTH FT-IN	MAXIMUM STROKE FT-IN	WIDTH BETWEEN JAWS (IN.)
33,000	33D	VULCAN	SINGLE	40-50	3,300	7,940	15'0"	10'0"	20
31,200	DA-55C	MKT	DOUBLE	78-82	5,000	17,000	17'0"	——	26
31,000-17,700	520	ICE	DOUBLE	80-84	5,070	12,545	16'6"	——	26
30,700-18,500	DA-45	MKT	DOUBLE	78-82	4,000	14,200	15'1	——	26
30,000	30D	VULCAN	SINGLE	40-50	3,000	7,630	15'0"	10'0"	20
29,000	B-225	BERMINGHAMMER	SINGLE	44-60	3,000	8,515	13'0	9'9"	——
28,100-16,550	MH15	MITSUIBISHI	SINGLE	42-60	3,310	8,400	16'1"	8'6"	26
28,050-19,800	DE-33	MKT	SINGLE	40-50	3,300	7,750	15'10¼"	10'0"	20
27,100-14,900	1500	F.E.C.	SINGLE	40-60	3,300	7,225	14'2"	10'11¾"	20
27,100	D15	DELMAG	SINGLE	40-60	3,300	6,600	13'11"	8'3"	20
26,200	B15	BSP	DOUBLE	80-100	3,300	9,000	17'0"	——	26
26,000-11,800	M14S	MITSUBISHI	SINGLE	42-60	2,970	7,260	13'7"	8'9"	26
25,200-16,800	DE30/20B	MKT	SINGLE	40-50	2,800	7,250	15'4"	10'0"	20
24,400	K13	KOBE	SINGLE	40-60	2,860	7,300	16'8"	8'6"	26
23,800-16,800	DA-35C	MKT	SINGLE	40-50	2,800	10,800	18'0"	13'3"	20
22,500	D12	DELMAG	SINGLE	42-60	2,750	6,050	13'11"	8'2"	20
21,000-16,000	DA-35C	MKT	DOUBLE	78-82	2,800	10,800	18'0		20
22,500-12,375	1200	F E C	SINGLE	40-60	2,750	6,540	14'0"	10'9¼"	20
18,100-7,700	440	ICE	DOUBLE	88-92	4,000	9,840	13'6"	——	20
18,000	312	ICE	DOUBLE	100-105	3,857	10,375	10'9"	——	26
18,000-10,000	B-200	BERMINGHAMMER	SINGLE	38-58	2,000	5,500	13'9"	10'2"	——
18,000-9,500	D8-12,22	DELMAG	——	38-52	1,770	4,760	——	——	——
17,000-12,000	DE30/20B	MKT	SINGLE	40-50	2,000	6,450	15'4"	11'6"	20
9,350-6,600	DA-15C	MKT	SINGLE	40-50	1,100	4,825	13'11"	10'6"	20
9,100	D5	DELMAG	SINGLE	42-60	1,100	2,750	12'2	8'3	19
8,800	DE10	MKT	SINGLE	40-50	1,100	3,100	12'2"	8'0"	10BP
8,200-6,600	DA-15C	MKT	DOUBLE	86-92	1,100	4,825	13'11"		20
8,100-4,060	180	ICE	DOUBLE	90-95	1,725	4,645	11'3"		20
39,400-18,900	D16-32	DELMAG	SINGLE	36-52	3,528	7,200	15'6"	11'2"	- -
58,250-29,500	D25-32	DELMAG	SINGLE	36-52	5,513	12,400	17'3"	10'7"	- -
70,000-35,400	D30-32	DELMAG	SINGLE	36-52	6,615	13,500	17'3"	10'7"	- -
84,000-41,000	D36-32	DELMAG	SINGLE	36-53	7,938	17,400	17'4"	10'7"	- -
107,000-52,200	D46-32	DELMAG	SINGLE	37-53	10,143	19,600	17'4"	10'7"	- -

HYDRAULIC VIBRATORY DRIVERS/EXTRACTORS

DYNAMIC FORCE (TONS)	MODEL	MANU-FACTURER	FRE-QUENCY VPM	AMPLI-TUDE IN.	HP	MAX PULL EXTRACTION TONS	PILE CLAMP FORCE TONS	SUSPENDED WEIGHT LB	SHIPPING WEIGHT LB	HEIGHT FT & IN	DEPTH FT & IN	WIDTH FT & IN
228	110H2	PTC	1,300	1 3/16	584	88	— —	26,600	48,600	9'9"*	2'11"	7'7"
228	110H1	PTC	1,300	1⅛	584	132	— —	38,200	60,200	7'1"*	2'6"	12'4"
204	1412	ICE	400-1,200	1-1½	650	80	250	20,400	41,000	12'8"	3'5"	8'0"
201	60H1	PTC	1,650	⅞	524	88	— —	29,100	50,100	8'11"*	2'3"	7'7"
182	V-36	MKT	1,600	¾	550	80	80	18,800	36,300	13'1"	1'0"	12'0"
160	V-30	MKT	1,600	1.00	510	80	110	15,000	32,500	9'9"	1'1"	— —
148	50H2	PTC	1,550	⅞	369	44	— —	32,450	38,750	10'2"*	1'1"	7'10"
145.4	812	ICE	400-1,600	½-1	475	40	100	15,600	30,200	9'0"	2'0"	8'0"
111.4	4000	FOSTER	1,400	.72	299	40	100 200	18,800	32,300	9'10"	1'10"	9'10"
100.5	V-20	MKT	1,650	.66	295	40	75	12,500	23,900	5'3"	1'2"	
84	25H2	PTC	1,650	15/16	253	30/44	— —	9,430	20,430	5'11"*	1'1"	7'6"
78.3	V-16	MKT	1,750	.47	161	50	75	11,700	20,600	5'3"	1'2"	— —
71.0	V-14	MKT	1,500	.32	140	50	75	10,000	29,500	5'3"	1'2"	— —
65.2	416	ICE	400-1,600	¼-1	250	40	100	13,100	26,200	8'9"	1'10"	8'0"
64	25H1	PTC	1,450	15/16	203	30/44	— —	9,430	19,330	5'11"*	1'1"	7'6"
40.5	1700	FOSTER	1,400	.39	147	30	80 100	12,900	26,900	7'0"	1'10"	— —
42	13H1	PTC	1,650	1 1/16	111	22	— —	4,850	10,850	4'6"*	1'1"	4'7"
36.4	216	ICE	400-1,600	¼-¾	155	20	50	4,825	12,500	6'6"	1'4"	3'11"
35.2	1200	FOSTER	1,425	.34	85	20	60	6,700	11,670	5'0"	1'11"	— —
30.0	V-5	MKT	1,450	.50	59	20	31	6,800	10,800	5'4"	1'2"	— —
23	6H1	PTC	1,800	.75	47	22	— —	3,180	6,580	4'1"*	1'1"	3'8"
21.8	116	ICE	400-1,600	¼-⅝	102	20	50	4,200	4,200	5'9"	1'4"	3'9"

*Less Clamp

IMPACT EXTRACTORS

RATED ENERGY FT LBS.	MODEL	MANUFAC-TURER	CRANE PULL-TONS MIN-MAX	BLOWS PER MIN.	RAM WEIGHT LBS.	TOTAL WEIGHT LBS.	LENGTH FT.-IN.	BOILER HP. REQ'D ASME	AIR REQ'D CFM
12,000	HD-15	BSP	18/45	120	3,850	11,380	19'4"	38	450
5,600	HD-7	BSP	8/22	150	1,155	3,750	16'4"	23	250
3,615	P-14	DELMAG	10/25	135	1,630	5,450	10'4"	DIESEL	— —
2,000	HD2000	BSP	3/20	200	500	1,680	8'5"	— —	125
1,640	1200-A	VULCAN	10.5/150	530	1,200	9,200	12'7"	140	1,020
1,000	800-A	VULCAN	7/100	550	800	5,640	10'8"	100	740
1,000	E-4	MKT	/100	400	400	4,400	10'5"	30	550
700	E-2	MKT	/50	450	200	2,600	8'4"	30	400
500	400-A	VULCAN	3.5/50	550	400	2,850	9'4"	50	342
250	200-A	VULCAN	.75 25	550	200	1,500	7'10"	18	173

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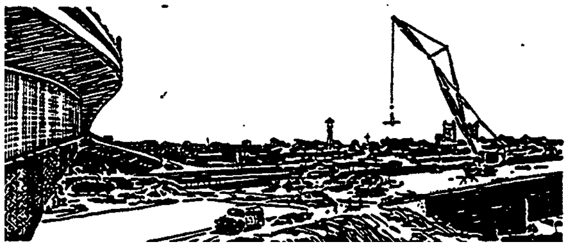

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Volume II

SETTLEMENT PLATFORMS

Introduction

Frequently it is necessary to construct an embankment or bridge approach fill on an unstable foundation material.

When this occurs, the foundation material is loaded for a specified period of time, or settlement period, prior to start of construction of the bridge. The loading consists of an embankment constructed to specified limits.

A settlement platform is used to evaluate the magnitude of the settlement of the embankment, and to determine the time when settlement ceases and the underlying foundation becomes stable.

Responsibility

The determination of the need to pre-load the foundation area, and the determination of the specified length of settlement periods for fills under bridge abutments is the responsibility of the Engineering Geology Branch.

The Transportation Laboratory furnishes and supervises the installation of the settlement platforms and associated instruments.

The reading of the instruments at the project level, and the determination of when settlement periods are to be terminated, are the responsibility of the Structure Representative. The Engineering Geology Branch will provide assistance in making this determination if requested to do so by the Structure Representative.

Requirements

The estimated settlement period duration will be set forth in the Special Provisions for the project.

Settlement platforms are required be neat embankments-having. a specified settlement period in excess of thirty (30) days, unless advised otherwise.

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As specified in the Special Provisions, a Contract Change Order will have to be written to permit compensation to the Contractor for labor, materials and equipment necessary to assist the Transportation Laboratory personnel with the installation of settlement platforms.

As soon as possible after the start of the work, the Structure Representative should consult the Constractor's construction schedule to determine when installation of the settlement platforms is required.

As soon as a tentative date for installation has been determined, the Structure Representative should so inform the Transportation Laboratory Foundation Section in Sacramento. This may be accomplished verbally or by letter through the Resident Engineer. This date should be confirmed as soon as a positive date has been determined.

Construction Control

After installation, reading of the instrumentation is normally performed by Structure field personnel. At regular intervals the data is recorded on special manifold forms which are supplied by the Transportation Laboratory. All necessary calculations are completed on the project and copies set to Engineering Geology Branch.

The frequency of reading the devices will vary according to circumstances involved in individual projects. In general, readings will range from one to three times a week during construction of the embankment.

After construction of the embankment, and during the settlement period, readings should be made at least twice a week. If the rate of settlement is relatively rapid, more frequent readings are in order.

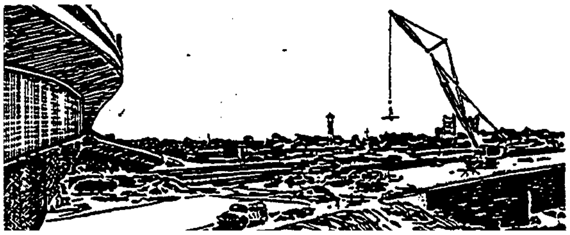
Termination of Settlement Period

As previously stated, the estimated settlement period is shown in the special provisions and the exact duration of the required settlement period will be determined by the Engineer.

At such time as the rate of settlement begins to decrease, the Structure Representative should contact the Engineering Geology Branch and keep them informed of the progress. His decision to terminate the settlement period should be concurred in by the Engineering Geology Branch.

The duration of the required settlement period at each location will be determined by the Engineer and will not exceed the number of days shown in the settlement data, except that when indicated in the Special Provisions, the Engineer may order an increase or decrease in the estimated settlement period. Such increase or decrease in any settlement period will result in an increase or decrease in the number of working days allowed for the completion of the work if the settlement period involved is considered to be the current controlling operation. Neither the Contractor nor the State will be entitled to any compensation other than an adjustment of contract time due to increases or decreases in the settlement periods.

Any decision to terminate, increase or decrease the settlement period should be transmitted in letter form to the Contractor, through the Resident Engineer, with a copy to the Office of Structure Construction.



SEAL COURSES

Where seal courses are shown on the plans, Section 51-1.10 of the Standard Specifications authorize the construction of a seal course only, "if conditions render it impossible or inadvisable in the opinion of the Engineer to dewater the foundation before placing concrete."

Since the purpose of a seal course is to seal the bottom of a tight cofferdam against hydrostatic pressure, the seal must be a construction necessity under the particular conditions encountered. Except as noted in Section 19-3.04 of the Standard Specifications, a seal is not to be used merely for the Contractor's convenience or to suit the Contractor's choice of construction methods.

Where seal courses are shown on the plans, the thickness (T) of seal courses is determined as follows:

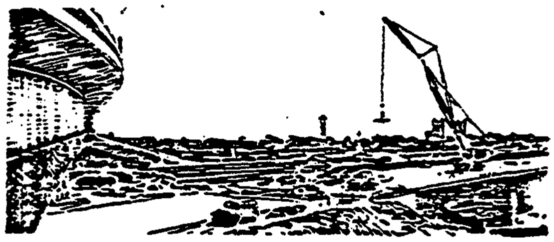
(a) For footings without piles, T = 0.43 H, but not less than 2'-0", unless otherwise shown on the plans. (H = hydrostatic head)

(b) For footings with piles spaced at 3-foot centers or less, T = 0.10 H + 1.0', but not less than 2'-0".

(c) For footings with piles spaced at 4' centers, T = 0.15 H + l.0', but no less than 2'-0".

(d) For footings with piles spaced on substantially more than 4-foot centers, discuss with area construction engineer or refer to office.

A note on the contract plans will indicate the elevation at which the bottom of footing is to be, constructed, in the event that the seal course is eliminated.



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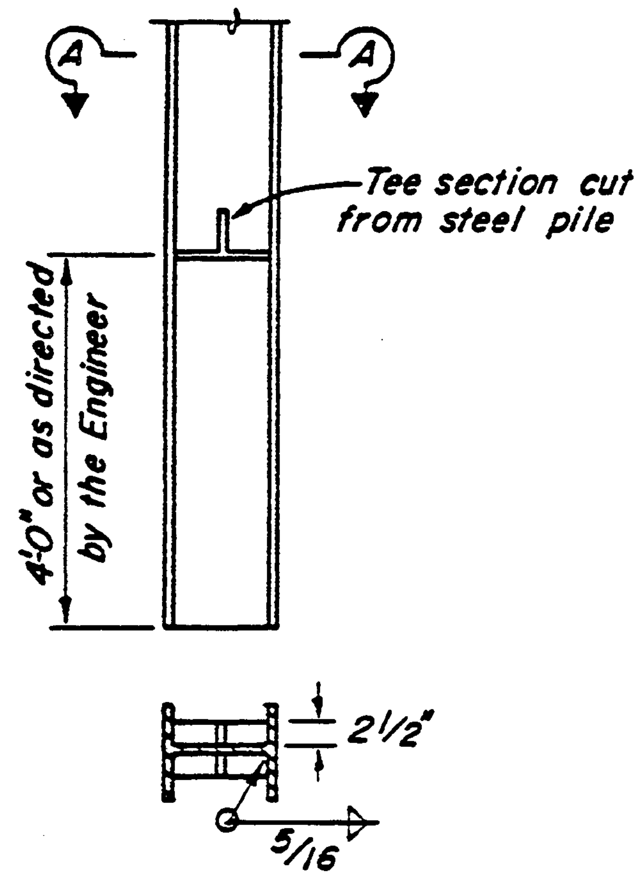
STEEL PILE LUGS

The "pile lug" detail for steel piles will be shown on the contract plans only when specifically required by the Engineering Geology Branch. When the "pile lug" detail is shown on the contract plans, it is mandatory that it be used. Full compensation for furnishing and installing the "pile lugs", as shown on the plans, is included in the contract price paid per linear foot for furnishing piling.

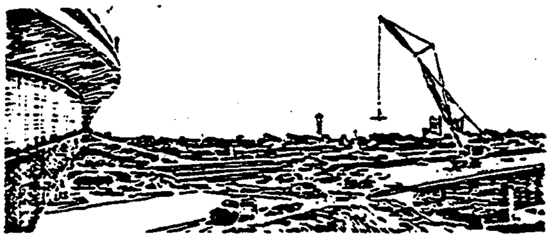
For contracts which do not show the "pile lug" detail on the contract plans, if the steel piles penetrate considerably deeper than the specified tip elevation, the Structure Representative may determine that it is advantageous to reduce the length of steel piling to be paid for by making use of "pile lugs". The details for "pile lugs" are shown on Attachment No. 1 of this Bridge Construction Memo. When the Structure Representative orders the Contractor to use "pile lugs", a Contract Change Order must be prepared to reimburse the Contractor on a Force Account basis for doing the work of furnishing and installing the "lugs".

If approved by the Structure Representative, the "pile lug" detail shown on Attachment No. 1 may also be used in the event that the Contractor elects to substitute steel piles for concrete piles. Such use should be permitted only if it is found that the steel piles tend to penetrate considerably below the specified tip elevation. Under these circumstances, the cost of furnishing and installing the "pile lugs" must be borne by the Contractor.



SECTION A-A

PILE LUG



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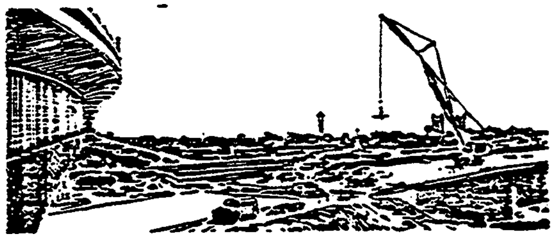
PAY FOR DRIVEN PILES

In connection with driving piles, Contractors frequently elect to avoid the cost of cutting off piles by driving the pile head to the required cut-off elevation. Driving additional lengths of pile, after the pile has attained the specified tip elevation and specified bearing, is considered to be of no benefit to the State. Therefore, there will be no payment for additional length of piling beyond that required to meet the contract specifications. If the Contractor elects to continue driving piles after contract requirements are met, he should be notified by letter that the additional driving and additional length of piling are at his expense.

Care should be taken to prevent damage to the piles due to the additional driving.



PAY FOR CAST-IN-DRILLED-HOLE CONCRETE PILING

Current Standard Special Provisions require that "The length of each cast-in-drilled-hole concrete pile to be paid for shall be the length, measured along the longest side, from the tip elevation shown on the plans or ordered by the Engineer to the plane of pile cut-off."

Unless the Engineer orders the Contractor, in writing, to change the cast-in-drilled-hole pile tip elevations, the Contractor will be required to drill the CIDH piles to penetrate at least to the tip elevations specified on the plans.

All cast-in-drilled-hole piles should be measured to determine the actual length and tip elevation.

Should the Contractor, through error, or by choice, construct the CIDH piles to have tip elevations lower than specified on the contract plans, the additional depth of pile shall be considered to be of no benefit to the State, and the Contractor will be allowed no payment for that length of CIDH pile below the specified tip elevation.

In order to avoid a controversy regarding the pay quantities, the Contractor should be advised of the aforementioned policy, in writing, in advance of beginning the pile drilling operation.

Volume II

BRIDGE CONSTRUCTION MEMO 130-9.0

FOUNDATIONS

October 19, 2001

MITIGATION OF CIDH PILES CONSTRUCTED USING THE SLURRY DISPLACEMENT METHOD

INTRODUCTION

Occasionally, CIDH piles constructed using the slurry displacement method need to be repaired, supplemented, or replaced before they can be accepted. This can occur if the Pile Acceptance Test Report, produced by Geotechnical Services, Office of Geotechnical Support, Foundation Testing & Instrumentation Branch (FT&I), reveals anomalies within the pile and the pile is rejected by the Engineer.

When a pile is rejected, the contract specifications require the Contractor to submit two documents to the Engineer for approval. The first document consists of "…written changes to the methods of pile construction…", hereinafter referred to as the **revised pile placement plan**. The second document consists of "…a plan for repair, removal or replacement of the rejected piling", or "…a mitigation plan for repair, supplementation, or replacement for each rejected cast-in-place concrete pile…", hereinafter referred to as the **pile mitigation plan**. No additional piles can be constructed until the Engineer has approved the revised pile placement plan. No rejected piles can be accepted until the Engineer has approved the pile mitigation plan and the mitigation work has been satisfactorily completed.

OSC PROCEDURE STATEMENT

When the Pile Acceptance Test Report reveals anomalies within a pile constructed using the slurry displacement method and the Engineer rejects the pile, <u>the rejected pile shall be mitigated only to the extent required for the pile to perform as intended by the design requirements.</u>

REVISED PILE PLACEMENT PLAN REQUIREMENTS

When the Engineer has rejected a pile, the Contractor is required to submit a revised pile placement plan prior to construction of any additional CIDH piling using the slurry displacement method. The purpose of the revised pile placement plan is to avoid construction of additional piling with problems similar to those encountered in the rejected pile. The "revised pile placement plan" shall be in the form of an amendment to the Contractor's original pile placing plan, with a complete description of the revisions that will be made to the pile construction methods.

The pile placement logs and the Pile Acceptance Test Report can indicate the actions necessary to avoid having similar problems on future piles. The following are several examples.

?? If the pile placement logs and the Pile Acceptance Test Report indicates the presence of pile "belling" or "necking", modifying the slurry viscosity and the length of time the hole is open may be appropriate.
?? If the Pile Acceptance Test Report indicates that the pile has a "soft" tip, revised bottom-of-hole cleaning procedures may be appropriate.
?? If the Pile Acceptance Test Report indicates that the pile has anomalies above the bottom of the pile, modifying the slurry viscosity and shortening the length of time necessary to place concrete may be appropriate.
?? If the Pile Acceptance Test Report indicates anomalies at the top of the pile, revised removal procedures of wasted concrete may be appropriate.

PILE MITIGATION PLAN REQUIREMENTS

Because of the inherent difficulties involved with evaluating rejected CIDH piles constructed using the slurry displacement method, the Division of Engineering Services (DES) has established the CIDH Pile Mitigation Plan Review Committee to assist the Structure Representative. Members of the DES CIDH Pile Mitigation Plan Review Committee include representatives of the Offices of Structure Construction (David Keim, phone (530) 587-0696, fax (530) 582-9106), the Office of Geotechnical Support, Foundation Testing & Instrumentation Branch (Brian Liebich, phone (916) 227-7164, fax (916) 227-7244), and the project design engineer.

To ensure uniform contract administration, procedures for administering the mitigation of rejected CIDH piles constructed using the slurry displacement method have been developed, which are contained herein. **The following are instructions to Structure Representatives regarding their responsibilities for investigation, plan review, and acceptance of rejected CIDH piles constructed using the slurry displacement method.**

There are four phases that a Structure Representative must complete during the process of mitigating a rejected pile. These include the Pile Investigation Phase, the Pile Mitigation Plan Development Phase, the Pile Mitigation Plan Review Phase, and the Pile Acceptance Phase. A flowchart describing the responsibilities for pile mitigation is included in Attachment No. 1.

Pile Investigation Phase

The Pile Investigation Phase begins when FT&I notifies the Structure Representative in writing that a CIDH pile should be rejected. This notification will be in the form of a Pile Acceptance Test Report. The Structure Representative shall immediately notify the Contractor in writing of any CIDH piling that has been rejected. A sample letter is included in Attachment No. 2.

There are five issues that the Structure Representative must resolve during this phase. Issue #1 is: **Does the rejected pile require mitigation?** Issue #2 is: **If mitigation is required, what are the design requirements for the rejected pile?** Issue #3 is: **What is the nature and extent of the anomalies in the rejected pile?** Issue #4 is: **Is it feasible to repair the rejected pile?** Issue #5 is: **What needs to be discussed with and transmitted to the Contractor?** Dealing with these five issues in a timely manner is essential.

An exception can be made for "simple" pile repairs. For rejected piles where "simple" repairs can be made, it will not be necessary to resolve these five issues, or to send the Contractor's pile mitigation plan to the DES CIDH Pile Mitigation Plan Review Committee for review and approval. A "simple" pile repair is a repair proposed by the Contractor to remove the soil around the defective portion of the pile, and remove and replace the defective portion of the pile. This type of repair can usually be performed when pile anomalies are reported within the upper 1.52 meters (5 feet) of the pile, where the defective concrete can be removed and replaced and the area of the pile to be repaired is visible. For proposed "simple" repairs for portions of the pile below the upper 1.52 meters (5 feet), other issues, such as disturbing the soil formation around the pile, need to be considered. Contact David Keim of the Offices of Structure Construction if you have any questions about what constitutes a "simple" repair.

For "simple" repairs, the specifications still require the Contractor to produce a mitigation plan. However, the Structure Representative does not need to go through the entire review process described herein, and may approve the plan, authorize the Contractor to proceed with the mitigation work, and accept the pile without any further review by the DES CIDH Pile Mitigation Plan Review Committee.

If a "simple" repair cannot be performed, the Structure Representative shall proceed with the pile mitigation process described herein. In order to resolve the five issues mentioned previously, the Structure Representative must rely on information from personnel in FT&I, the project design engineer, the project engineering geologist, personnel from Materials Engineering & Testing Services, Office of Testing & Technology Services, Corrosion Technology Branch (CTB), and the Contractor.

Issue #1: Does the Rejected Pile Require Mitigation?

To resolve Issue #1, the Structure Representative shall contact the project design engineer, the project engineering geologist, and Rob Reis of CTB at (916) 227-7287 to discuss the results of the pile acceptance test report. There are many factors, such as the location of the anomalies in the rejected pile, which may influence the response to Issue #1. However, there are only two possible responses to Issue #1. If the project design engineer, the project engineering geologist, the CTB representative and the Structure Representative review the results of the pile acceptance test report and determine that the rejected pile does not require mitigation, then the Structure Representative can proceed directly to the instructions for the response to Issue #5 of this Bridge Construction memo. Otherwise, the Structure Representative shall proceed to the instructions for the response to Issue #2 in the following paragraph.

Issue #2: If Mitigation is Required, What are the Design Requirements for the Rejected Pile?

To resolve Issue #2, the Structure Representative shall fax a copy of the Pile Design Data form (refer to Attachment No. 3 for an example) to the project design engineer and the project engineering geologist. A Pile Design Data form will be included in the Pile Acceptance Test Report produced by FT&I for each pile recommended for rejection. The Structure Representative shall discuss with the project design engineer and the project engineering geologist the two major areas of concern, which are structural and geotechnical. They shall use this discussion to determine the prevalent area of concern and to determine what data needs to be included on the Pile Design Data form.

Structural concerns:

If the rejected pile is designed to resist lateral or axial loads, the Structure Representative shall obtain the anticipated axial, bending and shear stresses at the location of the anomalies. This information shall be determined and entered on the Pile Design Data form by the project design engineer and faxed back to the Structure Representative.

Geotechnical concerns:

An explanation of how the forces transmitted to the rejected pile are transferred into the soil, whether by “skin friction”, “end bearing”, or some combinations of the two, must be provided by the project engineering geologist. Once these have been determined, the Structure Representative shall obtain the anticipated “end bearing” or “skin friction” forces at the location of the anomalies and other locations along the length of the rejected pile as necessary. This information shall be determined and entered on the Pile Design Data form by the project engineering geologist and faxed back to the Structure Representative.

The response to Issue #2 shall be considered to be complete as soon as the Structure Representative receives the completed Pile Design Data forms from the project design engineer and the project engineering geologist.

Issue #3: What is the Nature and Extent of the Anomalies in the Rejected Pile?

To resolve Issue #3, the Structure Representative must rely on the pile placement logs received from the Contractor and the expertise of the personnel at FT&I. Determining the nature and extent of the anomalies is the most difficult part of this procedure. Generally, the responsible engineer from FT&I will give an assessment of the anomalies in the initial Pile Acceptance Test Report. To further define the nature and extent of the anomalies, personnel from FT&I may perform cross-hole sonic testing of any pile that is recommended for rejection. The Structure Representative shall contact the responsible engineer from FT&I to discuss the results of the initial testing, the pile placement logs, and the results of the cross-hole sonic testing. From the results of this discussion, the Structure Representative and the responsible engineer from FT&I shall determine the assumed nature and extent of the anomalies in the rejected pile.

The Contractor may choose to perform their own investigation of a rejected pile in order to determine the nature and extent of the anomalies and to assist their development of the pile mitigation plan. However, the contract specifications do not require the Contractor to perform an investigation. **The Structure Representative shall not require the Contractor to perform an**

investigation of the rejected pile or use the results of any such investigation to reduce or eliminate the mitigation effort required on the part of the Contractor unless the Contractor performs their own investigation and requests the results be considered.

Issue #4: Is it Feasible to Repair the Rejected Pile?

To resolve Issue #4, the Structure Representative shall discuss the feasibility of repairing the rejected pile with the responsible engineer from FT&I, the project design engineer, and the project engineering geologist. Occasionally, the rejected pile is so badly damaged that it is not feasible to repair it. If it is agreed that repair of the rejected pile is not feasible, the rejected pile shall be supplemented or replaced.

If it is agreed that the rejected pile can be repaired, the Structure Representative and the responsible engineer from FT&I shall discuss the acceptance requirements for any mitigation work to be performed by the Contractor. Typical acceptance requirements include a “mitigation report” prepared by the Contractor, or additional pile acceptance testing performed by FT&I. The “mitigation report” is used to evaluate the effectiveness of the mitigation work after the mitigation work is completed. The “mitigation report” includes a description of the mitigation work as it was performed by the Contractor, along with grouting logs and other evidence that the mitigation procedures contained in the pile mitigation plan were adhered to and no further problems were noted. Occasionally, the pile will require additional acceptance testing, performed by FT&I, to verify that the mitigation work performed was acceptable.

Issue #5: What Needs to be Discussed with and Transmitted to the Contractor?

To resolve Issue #5, the Structure Representative must have resolved Issue #1, and Issues #2, #3 and #4, if appropriate. If the rejected pile does not require mitigation, the Structure Representative shall notify the Contractor in writing that no mitigative effort will be required and that the rejected pile can be accepted with no mitigation. A sample letter is included in Attachment No. 4. At this point, the Structure Representative can skip the “Pile Mitigation Development Phase”, “Pile Mitigation Plan Review Phase”, and “Pile Acceptance Phase” instructions contained herein and proceed to the “MEASUREMENT AND PAYMENT ISSUES” instructions contained herein. If the rejected pile requires mitigation, the Structure Representative shall hold a repair feasibility meeting with the Contractor to discuss whether or not the rejected pile can be repaired, or if it must be supplemented or replaced. If it was determined during the response to Issue #4 that it was not feasible to repair the rejected pile, the Structure Representative shall inform the Contractor that the rejected pile shall be supplemented or replaced.

The repair feasibility meeting does not have to be a formal meeting. For some types of pile mitigation, a phone conversation with the Contractor may be all that is necessary, once the Structure Representative has all of the information necessary to conduct the meeting.

As a follow-up to the repair feasibility meeting, the Structure Representative shall notify the Contractor in writing that pile mitigation will be required and the type of mitigation that will be permitted (repair, supplement, or replace) for each rejected pile. The Structure Representative shall include copies of the pile acceptance test report from FT&I, a copy of the filled-in Pile Design Data form for each rejected pile, and the cross-hole sonic test report if available, with the written notification to the Contractor. A sample letter is included in Attachment No. 5.

Rather than accept the results contained in the Pile Acceptance Test Report, the Contractor may choose to challenge the validity of the test results that are used to reject the pile. If the Contractor informs you of their intent to challenge the validity of the test results, contact David Keim in the Offices of Structure Construction for more information.

Pile Mitigation Plan Development Phase

The Pile Mitigation Plan Development Phase begins when the Structure Representative notifies the Contractor that the rejected pile requires mitigation.

Typical mitigation methods include installing replacement piling, installing supplemental piling to shore a rejected pile, or physically repairing the rejected pile. One method used to repair a rejected pile is to access the defective area of the pile by excavating the soil around the rejected pile. This usually is done for pile defects that are within 3 meters (approximately 10 feet) of the ground surface and includes "simple" repairs, although contractors have excavated to uncover pile defects that were much deeper. The most common pile repair method used to repair defective areas below 3 meters (approximately 10 feet) from the ground surface is pressure grouting, usually consisting of replacement, permeation or compaction grouting methods. Replacement grouting is generally used to repair void spaces within the limits of the pile that are known or are created by the Contractor as part of the mitigation process. Permeation grouting is generally used to repair defective areas where the nature of the defect is unknown within the limits of the pile. The nature of the defect could be a void space, an open "honeycomb" of softer material, or a zone of pervious material within the limits of the pile. Compaction grouting is used to enhance the load-carrying capacity of the soil around the pile. Compaction grouting is usually not necessary unless the rejected pile has been designed as an "end bearing" pile.

The Contractor is responsible for development of the pile mitigation plan. Newer versions of the specifications contain specific requirements for the pile mitigation plan. If the specifications do not contain specific requirements for the pile mitigation plan, as a minimum, the pile mitigation plan shall conform to the following requirements:

1. The designation and location of the pile addressed by the mitigation plan.
2. A review of the structural, geotechnical and corrosion design requirements of the rejected pile.
3. A step-by-step description of the mitigation work to be performed, including drawings if necessary.
4. An assessment of how the proposed mitigation work will address the structural, geotechnical and corrosion design requirements of the rejected pile.
5. Methods for preserving or restoration of the soil around the pile.

6. A list of affected facilities, if any, with methods and equipment for protection of these facilities during mitigation.
7. The State-assigned contract number, bridge number, full name of the structure as shown on the contract plans, District-County-Route-Postmile, and the Contractor (or Subcontractor if applicable) name on each sheet.
8. A list of materials and the estimated quantities of the materials to be used to perform the mitigation work.
9. Provisions for a "mitigation report", with a description of the contents, to be provided to the Engineer at the completion of the mitigation work.
10. For rejected piles to be repaired, the Contractor shall submit a pile mitigation plan that contains the following additional information:
    - 10.a. An assessment of the nature and size of the anomalies in the rejected pile.
    - 10.b. Provisions for preservation or restoration of the inspection tubes.
11. For rejected piles to be replaced or supplemented, the Contractor shall submit a pile mitigation plan that contains the following additional information:
    - 11.a. The proposed location and size of additional piling.
    - 11.b. Structural details and calculations for any modification to the structure to accommodate the replacement or supplemental piling.

Pile Mitigation Plan Review Phase

Upon receipt of the Contractor's pile mitigation plan, the Structure Representative shall review it and determine whether the pile mitigation plan conforms to the requirements stated in the Pile Mitigation Plan Development Phase or in the specifications. The Structure Representative shall return incomplete pile mitigation plans to the Contractor for correction and resubmittal.

If the Contractor has submitted a complete pile mitigation plan to replace the rejected pile, the Structure Representative shall review the pile mitigation plan with the project design engineer and the project engineering geologist to determine whether it is acceptable.

Once the Contractor has submitted a complete pile mitigation plan to supplement or repair the rejected pile, upon receipt of the Contractor's pile mitigation plan, the Structure Representative shall mail/fax/e-mail copies of the pile mitigation plan and the applicable filled-in Pile Design Data form to each member of the DES CIDH Pile Mitigation Plan Review Committee. The pile mitigation plan shall be reviewed by the DES CIDH Pile Mitigation Plan Review Committee and the Structure Representative to make sure it fully addresses the design requirements of the rejected pile's structural, geotechnical and corrosion deficiencies noted during the Pile Investigation Phase.

Responses to the Contractor's pile mitigation plan by the DES CIDH Pile Mitigation Plan Review Committee are coordinated through the Offices of Structure Construction. These responses will generally discuss whether enough information has been provided by the Contractor to evaluate whether the proposed mitigation method will resolve the structural, geotechnical and corrosion concerns. The Offices of Structure Construction will notify the Structure Representative in writing when the pile mitigation plan is acceptable. This notification will also include the acceptance requirements for the mitigated piling. A sample letter is included in Attachment No. 6. The Structure Representative shall not approve the Contractor's pile mitigation plan until this notice is received. Once this notification has been received, the Structure Representative shall notify the Contractor in writing that the pile mitigation plan has been approved.

Newer versions of the specifications include a time limit for the Engineer to review and approve the pile mitigation plan once a complete submittal has been received. For contracts utilizing newer versions of the specifications, the Structure Representative shall track the review time in a similar manner as with falsework submittals (refer to Section 2-1.05 of the Caltrans FALSEWORK MANUAL for details). In no case shall the time limit begin until a complete submittal has been received. Prompt turnaround of incomplete or inadequate pile mitigation plans by the Engineer and the Contractor is essential.

Pile Acceptance Phase

Once the pile mitigation plan has been approved, it is the responsibility of the Contractor to perform the mitigation work in accordance with the approved plan.

For rejected piles that are replaced, the Structure Representative shall not accept the replacement pile until personnel from FT&I have tested it and determined that it can be accepted (otherwise, the mitigation procedure starts all over again).

For rejected piles that are supplemented or repaired, the Structure Representative shall not accept the previously rejected pile until the Contractor submits evidence that the mitigation work performed is acceptable. The type of evidence required will be included in the notification to the Structure Representative from the Offices of Structure Construction. A typical form of evidence is the “mitigation report” described in the Pile Investigation Phase instructions. Another form of evidence would be the results of additional testing, performed by FT&I, to verify that the mitigation work performed was acceptable.

Newer versions of the specifications require the Contractor to prepare and submit a mitigation report to the Engineer for review and determination of whether the mitigation efforts were successful.

Once the evidence provided by the Contractor has been reviewed and accepted, or if the results of additional testing show the mitigation work performed is acceptable, the Structure Representative shall notify the Contractor in writing that the previously rejected piles have been accepted.

MEASUREMENT AND PAYMENT ISSUES

If mitigation for a rejected pile is required, the Contractor chose to or was required to replace the rejected pile, and the Structure Representative has accepted the replacement pile, the Structure Representative shall measure the length of the rejected pile and make full payment for the rejected pile only. Replacement piles shall not be paid for.

If mitigation for a rejected pile is required, the Contractor chose to supplement or repair the rejected pile, and the Structure Representative has accepted the mitigated pile, the Structure

Representative shall measure the length of the mitigated pile and make full payment for the mitigated pile only. Supplemental piles shall not be paid for.

If, as described in the Pile Investigation Phase instructions, mitigation for a rejected pile is not required, the Structure Representative shall notify the Contractor in writing that no payment shall be authorized for portions of the pile that are not mitigated or are deemed to be ineffective. A sample letter is included in Attachment No. 4. Portions of the pile that are not mitigated or are deemed to be ineffective by the project designer or engineering geologist always include the limits of the anomaly as described in the Pile Acceptance Test Report. However, portions of the pile deemed to be ineffective may also include portions of the pile below the top of the uppermost anomaly that was not mitigated. Offices of Structure Construction procedure is that any portion of the cross-section of the pile deemed to be ineffective shall project to the tip of the pile. Refer to the following paragraph for examples of this procedure. Unless otherwise specified, payment shall be withheld from the affected contract items of work. If the Contractor wishes to receive full payment for a rejected pile where mitigation is not required, the Contractor shall submit a pile mitigation plan and go through the mitigation procedure.

Newer versions of the specifications contain a method for calculating the amount of the deduction for an unrepaired pile. If the specifications do not include a method for calculating the amount of the deduction, the following examples of withholding of payment for ineffective portions of piles shall be utilized. Each example assumes a 1.2 m diameter pile, 20 m in length, with 5 inspection tubes.

1. An anomaly is present in the bottom 1.5 m of the pile, with the following criteria:
    ?? The Pile Acceptance Test Report states that the anomaly is present from the tip of the pile to 1.5 m above the tip of the pile and affects all 5 inspection tubes.
    ?? The Project Designer and Engineering Geologist agree that the bottom 1.5 m of the pile is ineffective. However, engineering analysis shows that the pile does not require mitigation.

? ? *The deduction is calculated as the prism defined by the cross-sectional area of the ineffective portion of the pile projected to the specified tip, multiplied by $400 per cubic meter, with a minimum deduction of $400. In this case, the volume would be calculated as ? *(1.2 m /2)² *100% *1.5 m * $400, or $678.58.*

2. An anomaly is present in the bottom 1.5 m of the pile, with the following criteria:
    ?? The Pile Acceptance Test Report states that the anomaly is present from the tip of the pile to 1.5 m above the tip of the pile and affects only 1 inspection tube, with a maximum of 20% of the cross-section of the pile affected within these limits.
    ?? The Project Designer and Engineering Geologist agree that 20% of the bottom 1.5 m of the pile is ineffective. However, engineering analysis shows that the pile does not require mitigation.

? ? *The deduction is calculated as the prism defined by the cross-sectional area of the ineffective portion of the pile projected to the specified tip, multiplied by $400 per cubic meter, with a minimum deduction of $400. In this case, the volume would be calculated as ? *(1.2 m /2)² *20% *1.5 m * $400, or $400.00.*

3. An anomaly is present between 6 m and 7 m from the pile cutoff elevation, with the following criteria:
    ?? The Pile Acceptance Test Report states that the anomaly is present between 6 m and 7 m from the pile cutoff elevation that affects all of the inspection tubes. Cross-hole sonic

logging was performed to further delineate the extent of the anomaly. The combined results indicate that 100% of the cross-section of the pile is affected within these limits.

?? The Project Designer and Engineering Geologist agree that 100% of the pile between 6 m and 7 m from the pile cutoff elevation is ineffective. However, engineering analysis shows that the pile does not require mitigation.

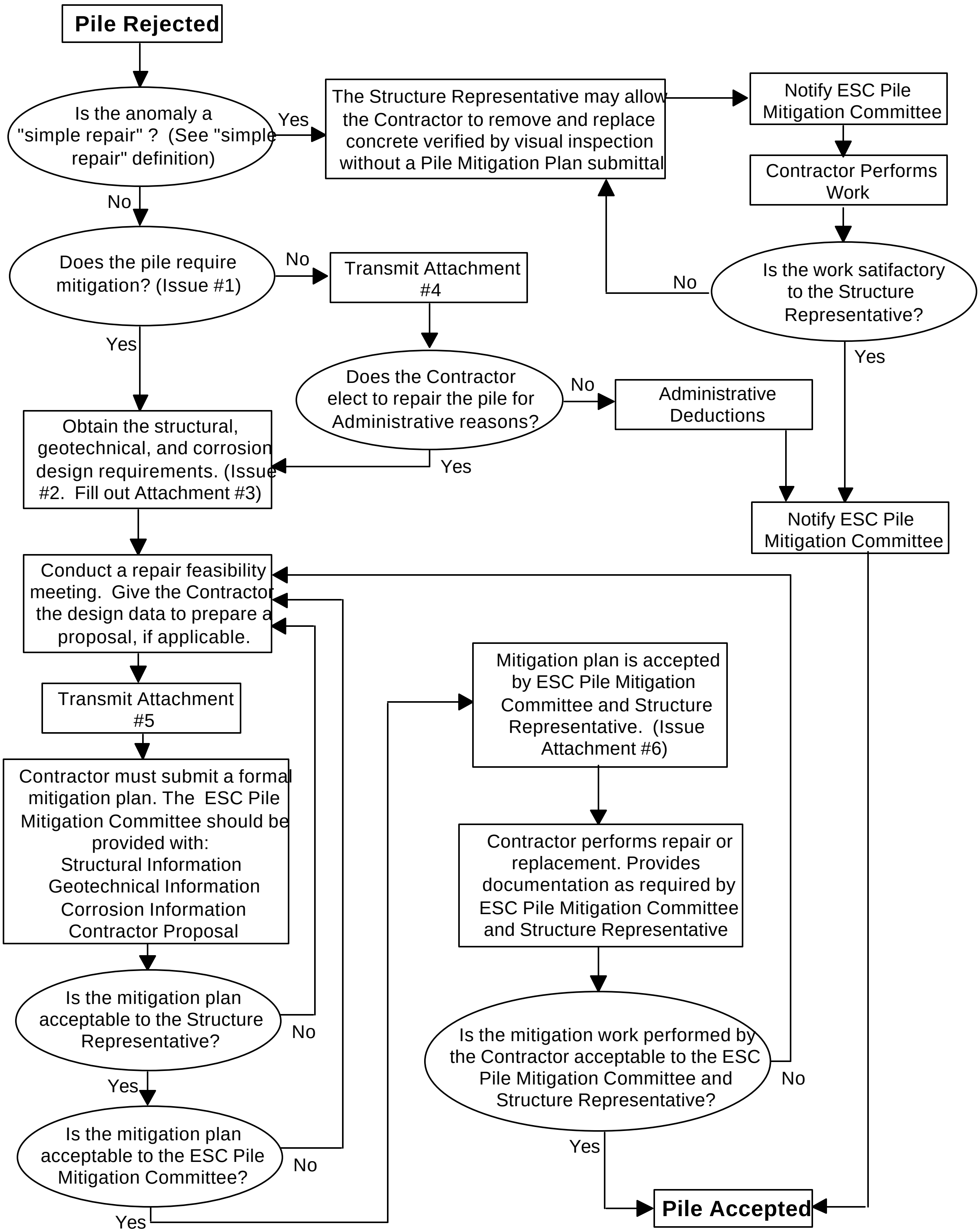
? ? *The deduction is calculated as the prism defined by the cross-sectional area of the ineffective portion of the pile* <u>*projected to the specified tip*</u>*, multiplied by $400 per cubic meter, with a minimum deduction of $400. In this case, the volume would be calculated as ? *(1.2 m /2)² *100% *14 m * $400, or $6,330.24.*

4. An anomaly is present between 1.5 m and 3 m from the pile cutoff elevation, with the following criteria:
   ?? The Pile Acceptance Test Report states that the anomaly is present between 1.5 m and 3 m from the pile cutoff elevation that affects 4 of the inspection tubes. Cross-hole sonic logging was performed to further delineate the extent of the anomaly. The combined results indicate that 25% of the cross-section of the pile is affected within these limits.

   ?? The Project Designer and Engineering Geologist agree that 25% of the pile between 1.5 m and 3 m from the pile cutoff elevation is ineffective. However, engineering analysis shows that the pile does not require mitigation.

? ? *The deduction is calculated as the prism defined by the cross-sectional area of the ineffective portion of the pile* <u>*projected to the specified tip*</u>*, multiplied by $400 per cubic meter, with a minimum deduction of $400. In this case, the volume would be calculated as ? *(1.2 m /2)² *25% *3 m * $400, or $2,091.24.*

Pile Mitigation Plan Procedure



DEPARTMENT OF TRANSPORTATION
<*Your office address*>
<*Your office phone*>

Date: <*date*>

File: <*job EA*>

<*Contractor name*>
<*Contractor address*>

Attention: <*responsible person*>

Dear Sir:

The attached CIDH pile acceptance test report for piles <*pile numbers*>, dated <*report date*>, has indicated the presence of anomalies in pile <*pile number*>, located at Bridge No. *xx-xxxx, <Bridge Name*>. Pile <*rejected pile number*> is hereby rejected in accordance with Section 5-1.09 of the Standard Specifications.

You are reminded of your responsibilities in Section 10-*xxx* of the Special Provisions, which require " ..written changes to the methods of pile construction..". before additional CIDH piling constructed using the slurry displacement method can continue. No additional CIDH piling constructed using the slurry displacement method shall be allowed until your revised pile placement plan has been received and approved by the Engineer.

An investigation is being performed to determine whether mitigation of pile <*pile number*> is required, and if so, whether pile <*pile number*> can be repaired or must be supplemented or replaced. You will be notified of the results of this investigation as soon as it has been completed.

Sincerely,

Resident Engineer

Attachment <*pile acceptance test report*>

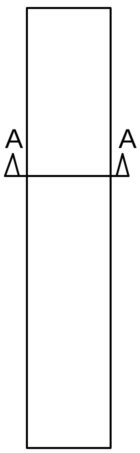
(A) Foundation Testing

Name: Brian Liebich
Phone: (916) 227-7235

Anomaly Overview

Testing Performed ☒ GGL ☐ CSL

Cutoff Elev.: 13.25 feet



Anomaly detected in 1 of 3 tubes from depth 32 to 33 feet by GGL

Tip Elev.: 71 feet

Anomaly Description

Section A-A: Anomaly Detected in Inspection Tube 2, affecting up to 33% of the pile cross section

Section B-B: ______________________________

(B) Structural Foundations or Geotech Oversight

Name: ____________
Phone: ____________

As Designed Resistance of Pile
Compression: ___________ Tension: ___________
GWT Elevation: ___________

Nomimal Axial Load at Anomaly A-A
Compression: ___________ Tension: ___________
Soil Type: ______________________________

Nomimal Axial Load at Anomaly B-B
Compression: ___________ Tension: ___________
Soil Type: ______________________________

(C) Structures Design or CCMB

Name: ____________
Phone: ____________

As Designed Strength for Pile Cross Section:
Shear: _________ Moment: _________

Factored Load of Pile at Affected Area as described in (A) at Section A-A:
Shear: _________ Moment: _________
Pile is structurally ☐ Adequate ☐ Inadequate with the anomaly in place at this depth.

Factored Load of Pile at Affected Area as described in (B) at Section B-B:
Shear: _________ Moment: _________
Pile is structurally ☐ Adequate ☐ Inadequate with the anomaly in place at this depth.

Comments: ______________________________

(D) Corrosion

Name: ____________
Phone: ____________

If Required

For anomalies between the top of the pile and 1 meter below the lowest possible ground water surface, California Test Methods (CTM's) CTM 643 (Parts 2, 3, 4 and 6 only), CTM 417, and CTM 422 are required to assess any proposed repair. For anomalies outside these limits, and where no stray current source can be identified, no consideration of corrosion potential is required.

Corrosion Potential At Anomaly A-A: ______________________________

Corrosion Potential At Anomaly B-B: ______________________________

Bridge Name: Carquinez Bridge	Bridge No.: 23-0015R	Support #: Bent D5
Dist/Co./Route: 04/CC,SOL/80	E.A.: 04-043933	Pile #: 1
Structure Rep.:	Phone #:	Fax #:

Caltrans

Pile Design Data Form

STATE OF CALIFORNIA -BUSINESS, TRANSPORTATION AND HOUSING AGENCY GRAY DAVIS, Governor{PRIVATE }

DEPARTMENT OF TRANSPORTATION
<Your office address>
<Your office phone>

Date: *<date>*

File: *<job EA>*

<*Contractor name*>
<*Contractor address*>

Attention: *<responsible person>*

Dear sir,

Please refer to my letter dated *<letter date>* regarding the rejection of pile *<pile number>*, located at Bridge No. *xx-xxxx*.

An investigation of the rejected pile performed by the Engineer has determined that no mitigation work is required and the rejected pile can be accepted as is. However, xx% of the cross-section of the pile between elevation *<elevation at the top of the ineffective section of the pile>* and elevation *<elevation at the bottom of the ineffective section of the pile>* is ineffective. Payment for contract items *<item numbers and descriptions>* for the ineffective section of the pile shall be withheld in accordance with Section 10-xxx of the Special Provisions if you choose not to perform any mitigation work. Full payment for contract items *<item numbers and descriptions>* for this pile will be made if you choose to properly mitigate the anomalies in the pile.

Please inform me whether you wish to accept reduced payment in exchange for acceptance of this pile, or if you wish to perform mitigation work on this pile in order to receive full payment.

Sincerely,

Resident Engineer

DEPARTMENT OF TRANSPORTATION
<Your office address>
<Your office phone>

Date: *<date>*

File: *<job EA>*

<*Contractor name*>
<*Contractor address*>

Attention: *<responsible person>*

Dear sir,

Please refer to my letter dated <*letter date*> regarding the rejection of pile <*pile number*>, located at Bridge No. *xx-xxxx*.

An investigation of the rejected pile performed by the Engineer has determined that mitigation work is required.

You are reminded of your responsibilities in Section 10-*xxx* of the Special Provisions, which require " ..a plan for repair, removal, or replacement of the rejected piling" before the rejected pile can be accepted. *<Structure Representative shall state whether a mitigation plan to repair the rejected pile will be allowed, or if the Contractor must submit a mitigation plan to supplement or replace the rejected pile>*

<If the specifications do not include pile mitigation plan requirements, add the following paragraph> Minimal requirements for the pile mitigation plan shall be as follows:

1. The designation and location of the pile addressed by the mitigation plan.
2. A review of the structural, geotechnical and corrosion design requirements of the rejected pile.
3. A step-by-step description of the mitigation work to be performed, including drawings if necessary.
4. An assessment of how the proposed mitigation work will address the structural, geotechnical and corrosion design requirements of the rejected pile.
5. Methods for preserving or restoration of the soil around the pile.
6. A list of affected facilities, if any, with methods and equipment for protection of these facilities during mitigation.
7. The State-assigned contract number, bridge number, full name of the structure as shown on the contract plans, District-County-Route-Postmile, and the Contractor (or Subcontractor if applicable) name on each sheet.
8. A list of materials and the estimated quantities of the materials to be used to perform the mitigation work.
9. Provisions for a " mitigation report" , with a description of the contents, to be provided to the Engineer at the completion of the mitigation work.
10. For rejected piles to be repaired, the Contractor shall submit a pile mitigation plan that contains the following additional information:
    10.a. An assessment of the nature and size of the anomalies in the rejected pile.
    10.b. Provisions for preservation or restoration of the inspection tubes.

11. For rejected piles to be replaced or supplemented, the Contractor shall submit a pile mitigation plan that contains the following additional information:
    11.a. The proposed location and size of additional piling.
    11.b. Structural details and calculations for any modification to the structure to accommodate the replacement or supplemental piling.

Attached is a copy of the original pile acceptance test report, the cross-hole sonic pile test *report <if available>*, and the pile design requirements to aid you in the preparation of the pile mitigation plan.

Please submit a pile mitigation plan to this office for review and approval as soon as possible.

Sincerely,

Resident Engineer

Attachments <*pile acceptance test reports, Pile Design Data form*>

State of California **Business, Transportation and Housing Agency{PRIVATE }**

M e m o r a n d u m

To: <*Structure Representative*>
STRUCTURE REPRESENTATIVE

Date: <*date*>

File: <*proj EA*>

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
OFFICES OF STRUCTURE CONSTRUCTION

Subject: CIDH PILE MITIGATION PLAN APPROVAL

The CIDH pile mitigation plan, dated <*date*>, submitted for pile <*Abutment/Bent number, pile number*> at the <*Bridge Name, Bridge Number*> has been reviewed and appears to be satisfactory.

<*CONTINGENCIES PARAGRAPH - If the mitigation plan is approved contingent upon anything, list it here.*>

<*CRITERIA FOR PILE ACCEPTANCE PARAGRAPH - Some type of criteria will generally be required after the mitigation work is completed to show the mitigation worked. Items 1. and 4. can be used alone. Item 3. may need to be included if Item 2. is used.*>
<*1. If additional testing is required, add the following:*> Upon completion of the mitigation work, the pile shall be retested by Geotechnical Services, Office of Geotechnical Support, Foundation Testing & Instrumentation Branch, prior to acceptance. Please make arrangements with the Foundations Testing & Instrumentation Branch for this testing.
<2. *If a mitigation report is required, add the following:*> Upon completion of the mitigation work, the Contractor shall provide a mitigation report to the Structure Representative for review prior to acceptance of the rejected pile. Acceptance of the rejected piles shall be contingent upon satisfactory completion of the pile mitigation work, as indicated in the mitigation report.
<3. *If the DES CIDH Pile Mitigation Plan Review Committee needs to review the mitigation report, add the following:*> The DES CIDH Pile Mitigation Plan Review Committee will also need to review the Contractor' s mitigation report prior to acceptance of the pile. Upon receipt of the Contractor' s mitigation report, please fax a copy to each member of the DES CIDH Pile Mitigation Plan Review Committee.
<*4. If nothing is required to accept the piles, add the following:*> Upon satisfactory completion of the mitigation work, the Structure Representative can accept the rejected pile.

If you have any questions about this, contact David Keim at (530) 587-0696.

David R. Keim
Senior Bridge Engineer

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 130-10 FOUNDATIONS**
Approved:____________________ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: July 1, 1999** **Expires: None** **Supersedes: None**

Subject: CIDH Pile Information Submittal

The Office of Structure Construction maintains a database that contains statistical data on CIDH piles constructed using the slurry displacement method. This database is used to evaluate the effectiveness of the contractors who perform this work and the effectiveness of the products provided by the various slurry manufacturers. **Structure Representatives shall assist in the acquisition of data by filling out the "CIDH Pile Information" memorandum for all CIDH piles constructed under slurry (or when gamma-gamma testing is required) and returning it to the Office of Structure Construction in Sacramento**. Refer to Attachment No. 1 for a sample of this memorandum.

A copy of Attachment No. 1 will be included with each Pile Acceptance Test Report. Do not return this memorandum to the Office of Structure Construction in Sacramento until all piles referenced in the Pile Acceptance Test Report (even those that require mitigation) have been accepted.

Attachments

cc: BCR&P Manual Holders
Consultant Firms

State of California | Business, Transportation and Housing Agency

Memorandum

To: RALPH P. SOMMARIVA, CHIEF
OFFICE OF STRUCTURE CONSTRUCTION

Date:

Attention: D. Keim

File:

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
OFFICE OF STRUCTURE CONSTRUCTION

Subject: CIDH Pile Information

The following additional information is provided for the CIDH piles referenced in the CIDH Pile Acceptance test report dated _________________.

Bridge number :____

Prime contractor :____

Drilling contractor :____

Pile No.	Date Pile Concrete was placed	Slurry Type: (Bentonite, SlurryPro CDP, SuperMud, Water)	Based on Pile Test Report, Pile was Rejected (R) Or Accepted (A)	If the Pile was Rejected, was Mitigation Required (Yes/No)	If Mitigation was Required, was a "Simple" Repair Performed? (Yes/No)	If Mitigation was Required, Mitigation was for Structural (S), Geotechnical (G), Or Corrosion (C) Concerns

Sincerely,

Structure Representative

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 130-11 FOUNDATIONS**
Approved: **R. P. SOMMARIVA, Chief Office of Structure Construction**	**Date: July 1, 1999** **Expires: None** **Supersedes: None**

Subject: Slurry Test Kits for CIDH Piles

The Office of Structure Construction has purchased slurry field test kits for verification of the slurry field testing performed by the Contractor for CIDH piles constructed using the slurry displacement method. To obtain a field test kit, contact your Bridge Construction Engineer. A list showing the original distribution of these slurry field test kits will be available on the OSC Intranet site.

These slurry field test kits will allow you to perform Density, Sand Content, Marsh Funnel Viscosity, and pH testing. Test methods for each of the slurry field tests required by the specifications are available in the CALTRANS FOUNDATION MANUAL, Appendix G. Slurry sampling and testing information is available in the CALTRANS FOUNDATION MANUAL, Chapter 9. It is recommended that slurry field testing be performed occasionally alongside the Contractor as a Quality Assurance measure.

There are several important items to verify when slurry field testing is performed:

- Prior to the start of construction of CIDH piles using the slurry displacement method, verify that the slurry sampler is capable of taking a discrete sample of the slurry at any level in the drilled hole as described in Chapter 9 of the CALTRANS FOUNDATION MANUAL.
- Check that the Contractor is taking slurry samples at the locations required in the specifications (at the mid-height and bottom of the drilled hole).
- When a slurry sample is taken, make sure that the contents of the slurry sampler are well mixed before slurry field testing is performed. This is especially important for the Sand Content and Density tests. Sand tends to settle to the bottom of the sampler very quickly, especially with synthetic slurries.
- Slurry field test results do not need to be the same at the mid-height and bottom of the drilled hole, but they do need to be within the requirements of the specifications at both locations.

In addition, a one-pint (approximately 500 ml) sample of the <u>synthetic</u> slurry used to construct CIDH piles should be sent to the Office of Materials and Foundations for chemical testing at least once per job. When filling out the sample testing ticket, request that an additional copy of the chemical test results be sent to: *Office of Structure Construction, MS-9, Attn: Substructure Committee Representative.* This chemical testing verifies that the synthetic slurry has the same chemical "fingerprint" as the synthetic slurry that was approved for use on Caltrans projects. The results of the chemical testing will be reported to the Structure Representative and to the Office of Structure Construction's representative of the Substructure Technical Committee. If problems are identified, the Office of Structure Construction will notify the Structure Representative. Chemical testing of <u>mineral</u> slurries is <u>not</u> required.

cc: BCR&P Manual Holders
Consultant Firms
JMStout, OMF

BRIDGE CONSTRUCTION BULLETIN **Approved:** **R. P. SOMMARIVA, Chief** **Division of Structure Construction**	**File: BCM 130-12** **FOUNDATIONS** **Date: January 14, 2000** **Expires: December 31, 2001** **Supersedes: None**

Subject: Approved Synthetic Drilling Slurries

In addition to the two previously approved synthetic drilling slurries (*SuperMud*, manufactured by PDS Co., and *Slurry Pro CDP*, manufactured by KB Technologies, Ltd.), a new synthetic drilling slurry has been approved for general use when constructing CIDH piling using the slurry displacement method. Approval of any synthetic drilling slurry is limited to sites that are <u>not</u> classified as primarily soft or very soft cohesive soils.

The newly approved synthetic slurry product is:

Product Name: *Shore Pac GCV*
Manufacturer: CETCO Drilling Products Group
Description: Dry granular powder

Admixtures that may be used with *Shore Pac GCV* include the following:

Product Name: Soda Ash (Sodium Carbonate)
Manufacturer: any
Purpose: pH conditioner

Product Name: *Insta-Floc*
Manufacturer: CETCO Drilling Products Group
Purpose: flocculant

Shore Pac GCV is similar in behavior and characteristics to the other two approved synthetic drilling slurries. This product may be used on any contract that contains language allowing the use of synthetic drilling slurries provided a <u>Contractor-requested</u> CCO is prepared permitting its use. For this CCO, the method of compensation shall be at Agreed Price, with a negotiated credit to the State. Language to be inserted into the CCO is attached. All other inspection guidelines and specifications shall remain in force.

If you have any questions about synthetic drilling slurries, cost issues associated with the CCO, or need verification of the proposed manufacturer's representative, contact David Keim at (916) 227-8814.

Attachment

cc: BCR&P Manual Holders
Consultant Firms

Substitution of *Shore Pac GCV* Synthetic Slurry Product

CCO Instructions:

You may write a CCO to allow the use of *Shore Pac GCV* synthetic drilling slurry on your contract. When writing the CCO, do not exclude any of the existing language in the Special Provisions. The following language shall be included in the CCO. The section of the Special Provisions referred to below as "Section 10-1.XX" shall be the section number referring to "Piling".

CCO Language:

Adjustment of Compensation at Agreed Price

In accordance with Section 10-1.XX of the Special Provisions, permit the use of Shore Pac GCV synthetic drilling slurry for construction of the Cast-In-Drilled-Hole piling.

Shore Pac GCV synthetic slurries shall be tested for conformance to the requirements shown in the following table:

Shore Pac GCV CETCO Drilling Products Group		
PROPERTY	REQUIREMENT	TEST
Density (kg/m^3) - prior to final cleaning - just prior to placing concrete	less than or equal to 1025*	Mud Weight (Density) API 13B-1 Section 1
Viscosity (seconds/liter) - during drilling - prior to final cleaning - just prior to placing concrete	35 to 78 less than or equal to 60	Marsh Funnel and Cup API 13B-1 Section 2.2
PH	8.0 to 11.0	Glass Electrode pH Meter or pH Paper
Sand Content (percent) - prior to final cleaning -just prior to placing concrete	less than or equal to 0.5	Sand API 13B-1 Section 5
*When approved by the Engineer, slurry may be used in salt water, and the allowable densities may be increased up to 32 kg/m^3. Slurry temperature shall be at least 4 degrees Celsius when tested.		

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 130-13 FOUNDATIONS**
	Date: August 27, 2003
Approved: ____________ **J.J. Abercrombie, Acting Deputy Division Chief Offices of Structure Construction**	**Expires: July 1, 2004**
	Supersedes: None

Subject: Pile Driving Acceptance Criteria

Projects advertised after April 30, 2003 with driven pile foundations specify the "Gates Formula" to determine nominal resistance. No longer will contracts utilize the ENR formula. This change is incorporated in the "Amendments To July 1999 Standard Specifications" (Section 49-1.08) found in the front of the Special Provisions. The Gates formula is:

$$R_u = (7 * \sqrt{E_r} * \log_{10}(0.83 * N)) - 550$$

Where:

R_u = Calculated nominal resistance in kilonewtons
E_r = Energy rating of hammer at observed field drop height in Joules
N = Number of blows per 300 millimeters

A simple spreadsheet (PileEquation-Gates.xls) used to calculated the N value can be found on the OSC Homepage under, "Downloads/Forms":

http://onramp.dot.ca.gov/hq/oscnet/

Background information regarding the change to the Gates formula can be found in the attached, "Replacement of Current Dynamic Formula for Pile Acceptance" issued by Geotechnical Services.

An error was made in the printing of some Contract's Specifications. CPD 03-04, issued September 2, 2003, addresses this issue with instructions for a sample change order. CPD's can be found at the following website:

http://projdel.dot.ca.gov/construction/CPDirectives/cpdindx.htm

Attachments

c: BCR&P Manual Holders
Consultant Firms

Memorandum

Flex your power!
Be energy efficient!

To: Geotechnical Committee

Date: September 24, 2002

File:

From: **DEPARTMENT OF TRANSPORTATION**
DIVISION OF ENGINEERING SERVICES
GEOTECHNICAL SERVICES - MS 5

Subject: Replacement of Current Dynamic Formula for Pile Acceptance

Background

As a result of the uncertainty of any Geotechnical design, methods of field verifying the adequacy of the design are required. The California Department of Transportation currently employs a simple dynamic formula known as the Engineering News Record (ENR) Formula for the field acceptance of most driven piles. This formula was first proposed in 1893 and has only been slightly modified in subsequent years to account for new types of driving systems. For the California Department of Transportation, the formula is contained in Section 49-1.08 of the Standard Specifications and determines the "safe load" that can be placed on a driven pile based upon observed behavior of the pile and pile driving hammer.

Limitations of the Engineering News Record Formula

The Engineering News Record Formula is widely recognized to have serious limitations and provide erroneous and potentially unsafe answers. The formula has been shown to be an inadequate method of pile prediction for 60 years. Some of the more important limitations are:

- The true factor of safety predicted by Engineering News has been shown to be as low as ½. As such, 12% of the driven pile foundation predicted to have adequate capacity by the ENR Formula actually possess a capacity less than the unfactored design load. The lack of capacity typically results in maintenance problems such as excessive settlements.

- Additionally, factors of safety as high as 20 have been documented. Extremely high factors of safety are wasteful of time and materials, and also lead to pile damage and therefore contractor claims.

- The ENR Formula possesses a larger associated variability in capacity prediction than the variability of the static analysis method utilized to design the piles. As such, Engineering News Record cannot adequately field verify the on-site geologic conditions. All piles are designed according to static analysis principles. Therefore, based on inherent precisions, it would be technically more accurate to not perform an ENR Formula prediction and use only the original design than to calculate and use the ENR value. For capacities predicted at the end of installation, the methods are normalized to the actual measured static capacity and compared below. The coefficient of variation associated with each method is the most important factor, as it describes the reliability of the method, with lower numbers indicating a high level of precision.

Prediction Method	**Mean**	**Coefficient of Variation**
Engineering News Formula	1.22	0.74
Static Capacity	1.30	0.68

- Little support and much opposition exists within the professional community for the continued use of the Engineering News Record Formula. The unmodified ENR Formula is no longer utilized by the Federal Highway Administration and 47 state highway transportation departments. California is therefore one of the last states to retain this formula.

- The Engineering News Record Formula uses unfactored "safe loads;" therefore it is not able to utilize load factor design methods. Additionally, the safe load system does not properly account for downdrag or the overburden effects and resistance associated with zones that may scour or liquefy.

Alternatives

While there are many limitations, the Engineering News Record Formula has survived primarily as a result of the simplicity of the formula and the familiarity of construction personnel its application. The California Department of Transportation recognizes the need to utilize pile capacity determination methods to perform field acceptance and confirm design. The primary alternatives to using the ENR Formula are the use of computer-based predictive models and the use of an alternate dynamic formula.

Use of Computer-Based Predictive Models. Computer-based predictive models, either from actual pile driving measurements, such as CAPWAP, or from soils and hammer information, such as wave equation, provide the best dynamic capacity predictions available. Of these methods, CAPWAP modeling is the most accurate method. For capacities predicted at the end of installation, the methods are normalized to the actual measured static capacity and compared below:

Prediction Method	Mean	Coefficient of Variation
Engineering News Formula	1.22	0.74
CAPWAP	0.92	0.22
Wave Equation (WEAP)	1.22	0.35
WEAP with Field Measurements	1.16	0.35

Unfortunately, each of these methods require intensive personnel training, specialized equipment, and time. The California Department of Transportation currently utilizes these computer-based predictive models for some of the critical structure sites, but not for projects involving standard plan piles. The quantity of trained engineers currently available to perform this type of modeling is severely limited, and the training process is lengthy. As such, while this may be an optimal solution for some piles, this is not likely to be an equivalent replacement for field acceptance in lieu of the ENR Formula.

Use of an alternate Dynamic Formula. In addition to the Engineering News Record Formula, numerous additional equations relating pile capacity with observed field conditions have been proposed. Of these, the Gates Formula has proven to be the most reliable method. This formula predicts the static capacity of the pile significantly more accurately than the ENR Formula. For capacities predicted at the end of installation, the two methods are normalized to the actual measured static capacity and compared below:

Prediction Method	Mean	Coefficient of Variation
Engineering News Formula	1.22	0.74
Gates Formula	0.96	0.41

Since the Gates formula possesses a significantly lower coefficient of variation than the Engineering News Record Formula, it is far more reliable. Additionally, since the formula utilizes ultimate capacity and not an unfactored safe load, the formula can account for the effects of downdrag, scour, and liquefaction. The Gates formula is also relatively simplistic and utilizes nearly the same input parameters as the currently utilized ENR Formula. The Gates formula is:

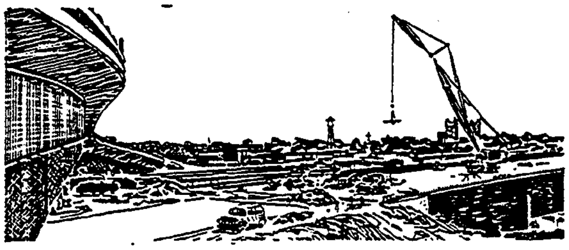
$$R_u = (7 * \sqrt{E_r} * \log_{10}(0.83 * N)) - 550$$

Where:
R_u = Calculated ultimate compressive capacity in kilonewtons
E_r = Energy rating of hammer at the observed field drop height
N = Number of blows per 300 millimeters

The Gates Formula can be utilized to calculate capacity or generate field inspector charts, identical to the current ENR Formula. Additionally, the Gates formula is the only dynamic formula currently recognized and accepted by the FHWA.

Recommendations

This Office recommends that the Engineering News Record Formula in the Standard Specifications be replaced by the Gates Formula. The Gates Formula provides superior precision and flexibility while retaining relative simplicity. This Office recommends that this change be implemented though Special Provisions on all future projects until the next edition of the Standard Specifications is published.



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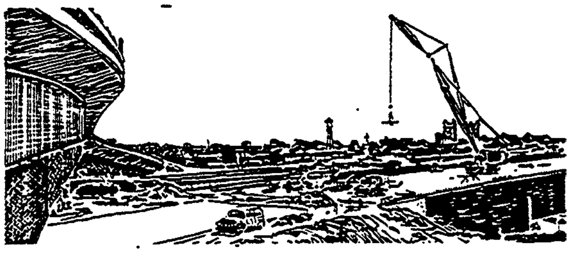
132-4.0	5-10-82	CHANGES INVOLVING BUILDING PROJECTS

A. P. BEZZONE, Chief
Office of Structure Construction

63F



WORKING DRAWINGS AND MATERIAL SUBMITTALS FOR BUILDINGS

The submittal of shop plans or equipment lists as specified in the Standard Specifications, the General Conditions, and/or the Special Provisions, requires that each submittal be completely identified. The Structure Representative should caution the Contractor early in the contract (this could be done at the pre-job conference) that this is a contract requirement, and that failure to comply will result in delay of approval of his submittals. (Attachment No. 1 to this Bridge Construction Memo is a sample of an equipment submittal which properly identified the equipment that the Contractor proposes to use.) The Contractor should also be informed that failure to make submittals that are not complete and not grouped in logical order also tends to delay the approval process.

The procedure for review and approval of working drawings and material submittals for buildings is a coordinated effort between the Architecture and Transit Branch, and Construction.

Attached is Memo to Architects A-5-11 (Attachment #2) "Review of Working Drawings and Material Submittal for Buildings". The design memo covers the procedures required for review and approval of working drawings and material submittals, including responsibilities of Structure Representatives on construction projects. Structure Representatives should comply with the applicable instructions in Memo to Architects A-5-1 in-so-far as possible.

Unless otherwise stipulated in the Special Provisions, the Contractor (subcontractor or fabricator) is to submit all working drawings and material submittals directly to the Office of Structure Design, Document Unit, P. 0. Box 942874, Sacramento 94274-0001. This includes original submittals and resubmittals. The Structure Representative is not to accept submittals unless it is so stipulated in the Special Provisions.

The working drawing and material submittals approval procedure is administered by the Special Services Group of Office of Structure Design. The group maintains a record of all working drawings and/or material submittals, and distributes copies to all interested parties, during all phases of the approval procedures. This relieves the Structure Representative of tedious administrative details necessary to insure that working drawings are distributed to the right people at the right time. One copy of all submittals will be forwarded to the Structure Representative on the same day that they are received in Sacramento.

The responsibility for checking working drawings and material submittals is shared by the Architect and the Structure Representative. Working drawings and/or material submittals shall not be returned to the Contractor until the Architect has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Architect and the Structure Representative.

Agastat® timing relays 7000 series

Contractor to show complete contract identification on each copy of each separate submittal. If submittals are not bound permanently in a folder, each sheet is a separate submittal.

BRIDGEPORT MAINT. STATION
09-MND-5722
CONT. NO. 09-056404

Model Number Code

TDR2
(REF. p. 88 SPEC. PROV.)

Contractor to completely identify item.

70 — AGASTAT® 7000 Series timing relay

1 — Operation
1 - On-Delay
2 - Off-Delay

2 — Contact Arrangement
1—Single Pole, Double Throw
2—Double Pole, Double Throw
① 4—Four Pole, Double Throw

A — Coil Voltage

	Coil Voltage
A	120 V 60 Hz 110 V 50 Hz
B	240 V 60 Hz 220 V 50 Hz
C	480 V 60 Hz
D	550 V 60 Hz
E	24 V 60 Hz
F	127 V 50 Hz
G	240 V 50 Hz
H	12 V 60 Hz
I	6 V 60 Hz
J	208 V 60 Hz
K	Dual Voltage (combines A & B)
M	28 VDC
N	48 VDC
O	24 VDC
P	120 VDC
Q	12 VDC
R	60 VDC
S	250 VDC
T	550 VDC
U	16 VDC
V	32 VDC
W	96 VDC
Y	6 VDC
Z	220 VDC

D — ③ Time Range

	Time Range
A	.1 to 1 Sec.
B	.5 to 5 Sec.
C	1.5 to 15 Sec.
D	5 to 50 Sec.
E	20 to 200 Sec.
F	1 to 10 Min.
H	3 to 30 Min.
② I	6 to 60 Min.
② J	3 to 120 Cyc.
K	1 to 300 Sec.

T — Aux. Switch Options
L
① LL
T

GZ — Optional Features
A —Quick Connect Terminals
①② B —Plug-In Connectors
①② C —Plug-In Receptacle (Screw Term.)
①② D —Plug-In Receptacle (Quick-connect Term.)
① GZ—Total Enc., W/Bottom Connection
①② H —Herm. Sealed (Consult Factory)
② M —Dustight
O —CSA Approval
① W —Watertight Enc., (NEMA-4)
② X —Panelmount Kit

OPTIONAL

① Not available on Panelmount models
② Not available on Four Pole models
③ Model 7014 available with letter-calibrated dial only.

Note: As shown above, the Contractor must show all options, accessories, and modifications to be furnished. Arrows, circles, or written notes may be used to identify the characteristics of the furnished item.

BRIDGE CONSTRUCTION MEMO 132-1.0
ATTACHMENT NO. 1 (10-21-81)
SHEET 1 OF 1

▌-Revised Since Last Printing

Review of Shop Drawings
and Material Submittal
for Buildings.

MEMO TO ARCHITECTS:

Procedure

The instructions in this memo apply to shop drawings and material submittals for buildings. Generally, this will apply to all projects prepared by Architectural Design.

Note:

The procedures covered in this Memo will also apply to Mechanical & Electrical (Building or related) projects. The responsibilities may be covered appropriately by inserting "M & E Engineer" whenever any reference is made to "Architect."

Structure Representative = Construction representative or Resident Engineer.

To provide uniform treatment in checking shop drawings and material submitted for buildings, the following procedure shall be followed:

1. The responsibility for checking shop drawings is shared by the Architect and the Structure Representative. Shop drawings shall not be returned to the Contractor until the Architect has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Architect and the Structure Representative.

   A brief file memo shall be written by the Architect to document controversial decisions or to keep other involved parties informed. For example, a memo is required for any change or clarification of details in contract plans. A copy of the memo is to be sent to the Structure Representative.

2. All submittals of shop drawings and materials will be received by the Documents Unit for distribution. The initial distribution of drawings will be:

   1 copy to RE or Structure Rep
   *4 copies to Architect

   *Including Mechanical/Electrical submittals, Structural submittals, and Landscape/Irrigation submittals.

   Architect will make distributions.

Replaces Memo to
Architects A-5-1 dated
September 1981,

The number of samples of each material may vary. The Documents Unit will submit all samples to the Architect, who will determine and record the disposition.

The Architect will check the drawings and:

(a) Sheets that do not requite correction. Al1 four copies will be stamped "Approved" and distributed as shown below.

PRINTS REVIEWED BY STATE OF CALIFORNIA **DEPARTMENT OF TRANSPORTATION** **OFFICE OF STRUCTURES – TRANSPORTATION ARCHITECTURE** **APPROVED** **PURSUANT TO SECTION 5–1.02 OF THE STANDARD SPECIFICATIONS** ____________ **DATE** ____________

1 copy will be retained by the Architect
3 copies will be returned to the Documents Unit who will make the following distribution:

1 copy to RE or Structure Rep
2 copies. to Contractor

(b) Sheets that have minor corrections. The Architect will stamp each of these sheets as shown and indicate all corrections in red on all four copies. The distribution will be the same as 2a.

PRINTS REVIEWED BY STATE OF CALIFORNIA **DEPARTMENT OF TRANSPORTATION** **OFFICE OF STRUCTURES – TRANSPORTATION ARCHITECTURE** **APPROVED** **SUBJECT TO NOTATIONS INDICATED IN RED** **PURSUANT TO SECTION 5–1.02 OF THE STANDARD SPECIFICATIONS** ____________ **DATE** ____________

(c) Sheets that have revisions. Only those sheets that require the corrections will be stamped as shown, Notes added to the sheet shall make it clear why the submittal is not approved. The distribution will be the same as 2a.

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED

PLEASE RESUBMIT

________________ DATE ________________

(d) Corrected copies received from the Contractor will be processed identical to the procedures outlined in 2, 2(a), 2(b) or 2(c) as required.

3. Far cantracts under General Conditions, the following stamps will be used under 2(a), (b), or (c):

(a)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED

PURSUANT TO SECTION 2-1.04 OF THE GENERAL CONDITIONS

________________ DATE ________________

(b)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED

SUBJECT TO NOTATIONS INDICATED IN RED
PURSUANT TO SECTION 2-1.04 OF THE GENERAL CONDITIONS

________________ DATE ________________

(c)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED
PLEASE RESUBMIT

__________ DATE __________

4. For contracts under General Specifications (under $25,000.), the following stamps will be used under 2(a). (b), or (c):

(a)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED
PURSUANT TO SECTION 5.02 OF THE GENERAL SPECIFICATION

__________ DATE __________

(b)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

APPROVED
SUBJECT TO NOTATIONS INDICATED IN RED
PURSUANT TO SECTION 5.02 OF THE GENERAL SPECIFICATION

__________ DATE __________

(c)

PRINTS REVIEWED BY STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
OFFICE OF STRUCTURES DESIGN

NOT APPROVED
PLEASE RESUBMIT

__________ DATE __________

Guide for Checking Shop Drawings

As a means of establishing uniform practice and avoiding omissions, but not as a substitute for common sense, the following outline is submitted as a general guide for the checking of the shop drawings. Some items are included which should fall within the duties of the Structure Representative. An overlap may avoid an oversight. These items will be reviewed in the discussion between the Architect and the Structure Representative.

If the individual Project Architect feels that certain factors need not be considered, or that others should be added, it is his prerogative to do so, providing the Structure Representative is agreeable.

1. Read the Standard Specifications (or General Conditions) and the Special Provisions for the particular job. The Special Provisions may modify the usual procedure. Read the correspondence file. There may have been changes approved by the Office of Construction since the contract was let. Call Structure Representative to establish a working relationship and to-become familiar with any pending changes or special problems.

2. Changes from the contract plans or specifications, regardless of magnitude, should not be allowed unless they have been discussed and approved by the Structure Representative and the Project Architect. Revisions may be satisfactory structurally or architecturally, but create administrative problems. Changes requiring Contract Change Orders as determined by the Structure Representative need special attention. These change orders could be grouped into two categories:

   (a) Those involving changes requested by the State and minor changes requested by the fabricator where there is no question on approval of the change order by both parties. The shop drawings can be approved, hut the note "Contract Change Order to be processed" must be added to each detail sheet involved,

   (b) Those involving changes requested by the Contractor, other than those covered in 2a. These should be returned to the contractor with the note "Request must be made by the contractor for a Contract Change Order." The Contractor may ask that the shop drawings be held by Architectural Design pending such negotiation. 'Architectural Design should not hold any plans without such a request.

3. All submittals shall be properly identified. This is covered in the Special provisions under Section 2-l.04 General Condition projects and 12-1.06 of Standard Specification projects. Bridge Construction Memo 132.1.0 instructs the Structure Representative to caution the Contractor of the importance of this identification (this could be done at the pre-job conference). Tabulated below are specific actions to be taken to assure proper identification:

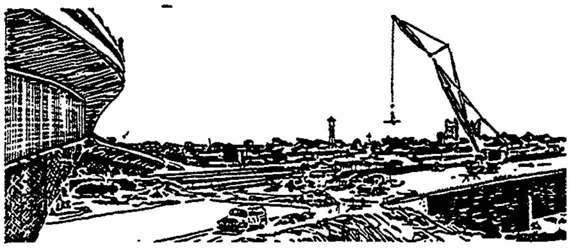
    a. If the project designer does not attend the pre-job conference be sure to notify the R.E. prior to the meeting to stress the importance of properly identifying shop plans.

    b. Inform the Contractor (via the Documents Unit) on the first returned transmittal of sets received with marginal identification. A call to the R.E. to discuss the problem is also in order at this time.

    c. Return seriously unidentified shop plans (via the Documents Unit) without checking (unstamped) when the practice continues after sufficient warning. This option should only be used as a last resort and only after getting the Senior's approval. When you exercise this* option a call to the R.E. and H.0. construction is mandatory.

Earl R. Latham

BRIDGE CONSTRUCTION MEMO 132-1.0
Attachment No. 2 (8-5-85)
Sheet 6 of 6



JOINT REVIEW OF BUILDING PROJECTS

Near the completion of a building project, the Structure Representative should arrange for a joint review of the project with representatives of other organizations who have a vested interest in the facility. The purpose of this review is to accomplish the following:

(1) Review the operation of the facility.

(2) Inform the Maintenance Regional Manager or the operators of the facility of the beginning date of the one year guarantee period and who to contact for guarantee work (this should also be covered in the transmittal letter required in Bridge Construction Memo 132-3.0).

(3) Discuss manufacture's warranties, service instructions, etc.

(4) Discuss work that may be required after contract acceptance.

(5) Review all design features that should be handled differently on future projects. These features should also be noted in the comprehensive letter which gives suggestions for improving the design or construction of building projects. (Refer to Bridge Construction Memo 2-8.0.)

The Structure Representative should arrange for the following to attend the review:

(1) Maintenance Regional Manager or his representative for building projects which will be operated and maintained by State Maintenance forces.

(2) A representative of the organization that will be operating and maintaining the facility for building projects not operated and maintained by State Maintenance forces.

(3) The project architect. The Architect will arrange for Headquarters representation at the review in accordance with instruction in the Transportation Architecture Manual (See Attachment No. 1).

At his discretion, the Area Bridge Construction Engineer may determine that minor construction projects do not warrant this joint review. Routine projects having a value under $35,000 such as Minor B contracts would fall into this category. If the review is not held, it is still required that input is obtained from Structure Design and that the appropriate people are informed about the operation of the facility and about the guarantee provisions and who to contact for guarantee work.

It is important that the Maintenance Regional Manager be kept informed regarding job progress on building projects which will be operated and maintained by State Maintenance forces. Therefore, he should be contacted prior to the start of the project work and encouraged to make periodic visits to the job site as the work progresses.

MEMO TO ARCHITECTS:

The attached Bridge Construction Memo 132-2.0 outlines a joint review upon completion of building projects. The Architect shall arrange for headquarters representation at this review. Typical representation would be:

Off ice of Business Management (OBM) Projects

Architect
Structures M & E Engineer(s)
OBM or H.Q. Maintenance Representation

Safety Roadside Rest Areas

Architect
Structures M & E Engineer(s)
Landscape Architect
Sanitary Engineer
H.Q. Maintenance Representation

Truck Weight & Inspection Stations

Architect
Structures M & E Engineer(s)
CHP Representative
Interagency Liaison with the CHP
H.Q. Maintenance Representation

Also attached is Bridge Construction Memo 132-3.0 that gives background on parts lists, service instructions, manufacturer's warranties and operating and maintenance instructions.

Earl R. Latham
Design Supervisor

Attachment

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 132-2.1 MISCELLANEOUS BUILDINGS**
Approved: R. P. SOMMARIVA, Chief **Office of Structure Construction**	**Date: May 1, 1998** **Expires: June 1, 1999** **Supersedes: None**

For

Subject: Inspection of Building-Related Transportation Facilities

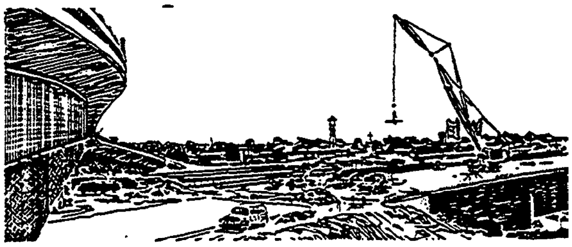
It is the Structure Representative's responsibility to ensure that all building-related transportation facilities work complies with the plans and specifications. However, some of this work is specialized and complex which the Structure Representative may not be totally familiar.

In order to be assured that the construction of building-related transportation facilities complies with the plans and specifications, the Structure Representative is encouraged to arrange a meeting between themselves and the ESC Project Design Team (Architect, Mechanical, Electrical, etc.) prior to start of construction of the facility. This meeting can be arranged by contacting either the Transportation Architecture Branch (Telephone 916-227-3962, calnet 498-3962) or the lead architect for the project as shown on the General Plan Sheet. This pre-construction meeting between the Office of Structure Construction, Transportation Architecture Branch and Electrical, Mechanical, Water and Wastewater Branch should be used to discuss clarifications to the plans or specifications and should be held prior to the pre-construction meeting with the Contractor.

A similar meeting between the above parties should be arranged at the completion of the project to review administrative and technical issues that may need refinement prior to future use of details, as well as procedures that worked well and should be considered for future projects. This meeting should be held in addition to the meeting required for the "Joint Review of Building Projects" as outlined in Bridge Construction Memo 132-2.0.

Structure Representatives are also referred to Bridge Construction Memo 115-l .0, Inspection of Electrical, Mechanical, Water and Wastewater Work.

C: BCR&P Manual Holders
Consultant Firms
RETravis, Transportation Architecture Branch
DLScharosch, Electrical, Mechanical, Water and Wastewater Branch
BGauger, Construction Program Manager



PARTS LISTS, SERVICE INSTRUCTIONS, MANUFACTURER'S WARRANTIES AND OPERATING AND MAINTENANCE INSTRUCTIONS

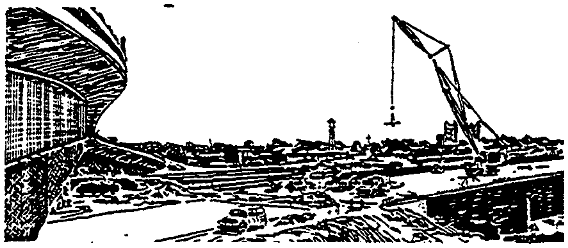
The special provisions for building projects generally require that certain operating and maintenance instructions be submitted in duplicate.

The special provisions for building projects also require that parts lists, service instructions and manufacturer's warranties for products installed in the work shall be delivered to the Engineer.

It is the Structure Representative's responsibility to see that the required parts lists, service instructions, manufacturer's warranties and operating and maintenance instructions are furnished by the Contractor.

If lists, warranties, or instructions are furnished in duplicate, one copy is to be sent to the District Maintenance Engineer and the other copy is to be given to the operators of the facility. If only a single copy is furnished, it should be given to the operators of the facility.

The parts lists, service instructions, manufacturers warranties and operating instructions, furnished to the operator of the facility, and/or to the District Maintenance Engineer, should be accompanied by a transmittal letter. This letter should list all of the instructions, warranties, or parts lists furnished, and give the name, address and phone number of the Prime Contractor. A copy of this letter should be forwarded to the Sacramento Structure Construction Office.



CHANGES INVOLVING BUILDING PROJECTS

Before making any changes involving building projects, the Structure Representative must contact the appropriate section of Structures Design, Architectural and Transit Branch, to inform them of the proposed change, and to obtain their concurrence for the proposed change. Contact with Structures Design may be made either in writing, by telephone, or in person.

When the Contract Change Order Letter of Transmittal is prepared, the Structure Representative must include the following information: Name of person contacted in Structures Design, the method of contacting the person in Structures Design, and a statement that the person contacted concurred with the necessity for, and the provisions of the proposed change.

See Bridge Construction Memo 7-2.0 for additional information concerning Structure Construction change order policy.

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Volume II

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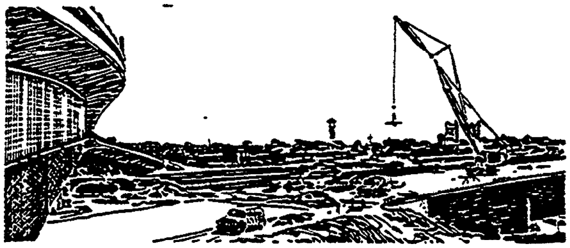
135-3.0	04-16-90	COMPRESSIVE DEFLECTIONS OF FIBERGLASS OR STEEL REINFORCED ELASTOMERIC BEARING PADS
135-4.0		(BLANK)
135-5.0	4-11-03	MECHANICAL ANCHORAGE DEVICES

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction



EPOXY ADHESIVES

General Information and Instructions for Use

Epoxy adhesives are very good construction materials. They are, however, somewhat complicated, so there are certain rules that must be followed when using them. Some of these rules are seemingly insignificant, but experience has shown that if they are not closely followed, failure is likely to occur. As a step toward reducing epoxy failures, the adhesive selection, mixing and placing requirements for good performance will be reviewed.

Epoxy adhesives in their pure form are very hard and extremely brittle, and have undesirable properties for structural use. These physical properties can be altered to fit a wide spectrum of hardness and flexibility by judicious selection of hardening and flexibilizing agents. It is only natural, then, that there is a correct epoxy adhesive formulation for each type of job requirement: high strength epoxy to resist high stresses, flexible epoxy to resist high thermal changes.

All State Specification epoxies are of the two component type. One component is the epoxy resin and the other is the hardening agent. To these components are added coloring, stiffening agents, and flexibilizers as required. Some epoxies are designed as 1:l mix ratio of the components, others as a 2:l. They are also designed to have a specific pot life at a certain temperature. When the two components are combined, there are two very important rules which must be followed: (1) mixing proportions as shown on the container must not be changed, (2) the components must be thoroughly blended. One should never try to alter the pot life of an epoxy by changing the prescribed mixing ratio. Doing so would result in a very undesirable epoxy. State Specification epoxies with designed pot lives from a few minutes to about 40 minutes are available. Hence, if the epoxy on hand is either too fast or too slow for a designated job, it should not be used; one which has a pot life that is more compatible with the job requirements should be obtained. The temptation to add solvents to reduce viscosity, extend working time or improve application characteristics must be avoided. Such usage could cause poor adhesion, high shrinkage and a "cheesy" effect.

The surface receiving the epoxy adhesive must be clean, sound, and dry. Probably the most frequently ignored requirement for a receiving surface is cleanliness. This is especially true when saw-cutting has been done. It is often assumed that a freshly sawed surface is clean since it has been continuously washed by the saw-blade cooling water, when actually a fine residue is left by the water. To insure a clean saw cut surface, it should be sandblasted before application of the epoxy. Cracked or loose sections of concrete near the surface should be located by striking the concrete with a hammer or by dragging. a chain over it. Hollow sounds are indicative of fractured concrete. Loose concrete must be removed.

In respect to strength and permeability, rules which have been developed for portland cement concrete are also applicable to epoxy concrete. The primary difference between the two concretes is that cement, with water for hydration, is the binder in one whereas epoxy is the binder in the other. Similar conditions produce similar results in each. For instance, a uniform gradation of sand and aggregate produces a stronger product than does a single or gap gradation; the richer the mix the less permeable the final product, etc. It follows then that regular concrete sand and gravel, thororoughly dried; usually produces a good epoxy concrete mixture: The maximum size aggregate, as in regular PCC, is determined by job conditions: Depth of section, reinforcing steel restrictions, etc.

The ratio of epoxy to aggregate for a strong, dense mix depends to a great extent on the gradation and maximum size aggregate used. Rule of thumb epoxy/aggregate-ratios (by volume) for good aggregate gradation of various maximum sizes are: l/5 for 1 inch l/4 for pea gravel, and l/3 for 2O mesh sand. Whenever possible, make sample mixes using different ratios of mixed epoxy and aggregates. By comparing these sample mixes, it will be possible to select the mixed epoxy/aggregate mix that is most appropriate to the work. Since the viscosity of epoxies varies considerably with temperature, the mixing ratio of mixed epoxy/ to aggregate may have to be adjusted to maintain workability whenever there is a drop in temperature. When making the sample mixes, use the same compactive effort that it is anticipated will be used when placing the mix on the job. A good mix design is evident if the sample exhibits a mixed epoxy rich surface when compacted in this manner. A mixed epoxy rich surface results when the mix contains more mixed epoxy than is necessary to coat the aggregate and fill the voids. If an epoxy patch is placed on an exposed deck, an epoxy rich surface will be dangerously slippery.

Required skid resistance can be obtained by broadcasting dry sand on the surface while the epoxy is still fluid enough to receive it.

In an epoxy concrete mixture, it is most important that the epoxy resin components be blended first and then thoroughly mixed into the sand or sand and aggregate. Sides and bottom of mixing containers should be scraped clean during the mixing operation. The mixing operation within the limitation imposed by the pot life must be completed and the mixed epoxy/aggregate placed and compacted before the pot life is exceeded. The results of improper mixing are: non-uniform curing, cheesy or sticky areas, brittle areas poor adhesion and poor performance. The sand and aggregate has to be clean and dry to ensure proper bonding by an epoxy adhesive. Often it is incorrectly assumed that since epoxy will successfully bond wet concrete to dry concrete, good epoxy concrete can be obtained with damp aggregate. This is a false assumption.

In an epoxy rich mixture, there appears to be enough free epoxy to provide bond to the concrete surface against which it is to be placed. However, experience has shown that this is not true. Therefore, to ensure proper bond, the surface must be primed with pure epoxy adhesive just prior to placing the epoxy aggregate concrete.

Epoxy concrete varies as its counterpart PCC in placing characteristics. Epoxy concrete is too sticky and viscous to be effectively vibrated with a spud type vibrator. Surface vibration with a flat plate has not been tried, but may work. The most effective proven way to place epoxy concrete is to work it around reinforcing steel and into corners by hand. Rodding it with a 1 x 1 helps, but generally the springing or bulking characteristic of the material makes rodding somewhat ineffective. Rodding is most effective after the material has been in place a few minutes and some bleeding of the pure epoxy has occurred.

When epoxy concrete is to be placed in thickness greater than 2" it should be placed and compacted in lifts of 2" or less thickness.

The surface of an epoxy concrete can be finished with either a wood float or steel trowel as desired. The steel trowel is superior to a wood float in sealing the surface of an epoxy mixture.

Temperature is a critical parameter in the curing of epoxy, the higher the temperature the faster the cure (and generally the higher the strength.) This fact becomes important when epoxies are used for patches or seals on decks and the controlling factor for opening the deck to traffic is curing of the epoxy.

Curing of epoxy can be accelerated by externally applied heat. Best results for placement in cold weather is to heat the concrete receiving surface preheat the epoxy components and aggregates before any mixing is done and then heat the mixture after it is placed. Preheating the individual components will probably significantly decrease pot life. Experimentally determine the reduction of pot life at the temperature of application. The in-place heating should not be done by direct flame onto the epoxy, but rather by radiant heaters, or by heated air such as is provided by heating a steel plate elevated above the epoxy surface, or by heating the inside of a "tent" erected over the work. Heat lamps directed towards the epoxy is also another good source of heat. The heat of the PCC surface at time of placement, heat of the components before mixing, or heat of the epoxy concrete after placing, should not be greater than about 110° F. When heating the PCC surface, a heater which will not contaminate the surface should be used.

When epoxy is used to bond fresh concrete to hardened concrete, the fresh concrete should be as dry as working condition will allow and must be placed while the epoxy is still fluid. If the epoxy reaches a firm but still tacky state, a new coat of epoxy must be applied onto the hardened concrete before the concrete is placed. If, on the other hand, the epoxy cures beyond the tacky state, it should be sandblasted before the new epoxy coat is applied.

Epoxy concrete dams at expansion joints or epoxy repaired joint spalls should be protected during the epoxy curing period from harmful pressures caused by joint closure as the structure expand. Easily compressed plastic foam materials placed in the joint provides good protection. Forms against which the epoxy concrete is to be placed shall be coated with paraffin or silicone grease, or covered with polyethylene sheet to prevent bond. The epoxy concrete should not be allowed to flow under the forms and encroach into the space reserved for joint closure. If the material is allowed to flow into this space, failure of the repair is certain when the joint closes against it.

Epoxy injection or pressure sealing cracks with epoxy can be done with 8040-OlE-01, 8040-01E-02, or 8040-0lE-03 material. However, most epoxy injection on contract jobs is done by a subcontractor specalist using epoxy he normally uses for injection work, The Transportation Laboratory should be consulted before permitting the use of other than State Specification epoxies for injection or any other type epoxy work.

The coefficient of thermal expansion for epoxy adhesives is roughly 5 times that of concrete. This large a difference can be tolerated for most epoxies down. to approximately 15° F. because they are still flexible enough at this temperature to "give" under the stress induced by volume change differences. However, since the flexibility of epoxies varies as the temperature, the coefficient difference becomes critical at lower temperatures, Without proper flexibility in the epoxy system, the differential volume change will usually cause shear failure in the concrete, which is generally weaker than the epoxy. There is still sufficient residual flexibility at the lower temperatures in the epoxies designed to bond new concrete to old when their, in-place thickness is l/8 inch or less. These epoxies, however, cannot be used in greater thickness, or in epoxy concrete, or in epoxy mortar; more flexible epoxies are available for these uses.

The exact plastic flow characteristics of epoxy is still undetermined. Hence, until more knowledge is gained on this subject, epoxies should not be used in a manner that will subject them to sustained axial loads.

Where there is no abrasion and a protective coating is required, such as area of edge of deck to drip groove and around scuppers, Design has been using an epoxy enamel such as found in Section 91-4.04 of the Standard Specifications.

Rules, regardless of how complete they may be, are effective only to the extent to which they are followed. In the epoxy use rules discussed, each is a vital link in the process which produces a successful job. Consequently, the degree of success, as measured by in-use performance, of an epoxy application is dependent on the attention given to the adhesive selection, mixing and placing requirements.

For additional information concerning epoxies, contact Tom Shelly (916) 739-2346, ATSS 497-2346, of the Transportation Laboratory.

Safety Precautions

The following precautions are to be followed by all field personnel who are involved with the use of epoxy resin materials:

> "The exposed parts of the face, neck and hands should be protected with barrier creams and plastic or rubber gloves be worn during the mixing, blending and placing operations.
>
> "When resins or solvents come in contact with the skin, it should be washed with soap and water, Do not use solvents to clean epoxy from the skin, use soap and water.
>
> "Goggles or face shields should be worn to prevent vapor or liquid splashes from coming in contact with the eyes. If uncured resins or solvents do come in contact with the eyes, they should be flushed continuously for ten minutes and then receive medical attention.
>
> "Contaminated clothing, rags, gloves, etc., should not be reused.
>
> "Good ventilation must be provided for the preparation and use of epoxy resin concrete; and since there entails a fire hazard, fire-fighting equipment must be maintained at all operations."

Following are listed seventeen safety rules which should be followed to offset the hazards inherent in the use of epoxy resins:

1. Inform workers of the hazards of epoxy resin operations and show them how to avoid contact.

2. Provide special isolated areas in the plant for mixing, molding, curing, casting, and tooling of epoxy resins.

3. Install ventilated hoods in mixing areas to prevent the spread of hazardous vapors.

4. Limit mixing to only a few workers.

5. Ventilate grinding, sawing, drilling or polishing operations where epoxy resins are used.

6. Supply protective sleeves and cotton liners under rubber gloves to workers in molding operations.

7. Provide water-soluble skin protective gels.

8. Supply neutral or acid soap instead of alkaline, powdered or abrasive cleansing agents.

9. Don't permit workers to use acetone or solvents to cleanse the skin.

10. Replace clean-up rags with disposable paper towels.

11. Institute a strong housekeeping program that immediately washes up spills and keeps tables, machinery, tools, floors, walls and windows free of particles and dust.

12. Provide goggles and respirators when epoxy resins are sprayed.

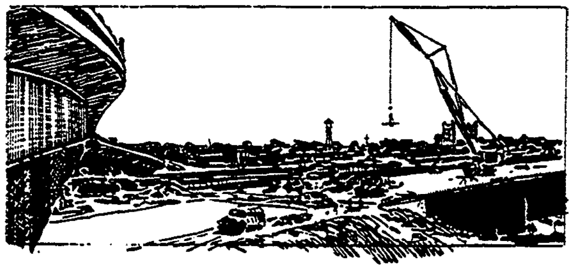
13. Cover benches and seats in mixing areas with disposable paper.

14. Throw away empty epoxy resin containers and drums.

15. Enforce a program of individual worker sanitation which requires washing before eating, before relief periods, after work, and after any contact with epoxy resins.

16. When possible, mechanize blending, mixing, and pouring operations.

17. Prohibit the wearing of clothing soiled by epoxy resins.



BRIDGE DECK EXPANSION JOINTS AND JOINT SEALS

CONSTRUCTION POLICY:

a. General

All deck expansion joints and joint seals, except for special cases, will be specified by seal type and M.R. (Movement Rating). The success or failure of joint seals will depend greatly on the enforcement of the specifications. Questions concerning joint seals will be handled in normal channels through the Construction Engineers and the Structure Construction Office.

It is the Structure Representative's responsibility to:

(1) Determine the proper groove width or installation width for the joint seal used, and to complete the applicable portions of the "Joint Movements Calculations" sheet (Form DS-D129).

(2) Install movement recording scribes on all expansion joints.

b. Special Details

Check details such as water stop, formed joint openings, hinge restrainers, rollers or rockers, conduits, etc., for proper setting and movement capacity. All components in an expansion joint must be capable of withstanding more than the anticipated movement for a particular joint.

Joints to be sealed under rehabilitation contracts must first be cleaned of all existing seal material, joint filler, dirt and debris to the top of the waterstop. If the joints do not have a waterstop, or the waterstop is damaged, it is essential that the joint be cleaned down to the bearing or hinge seat. Care should be taken so that existing utilities and encroachments spanning joints are not damaged by the cleaning operations. Carefully inspect the condition of the existing joint and the face of the saw cut. It may not be necessary to resaw cut the joint. If not, a change order may be written to eliminate the saw cutting with a credit to the State.

All dimensions of the existing joint must be verified to be compatible with the new seal, including the depth. All joint damage shall be repaired as directed by the Engineer. Sawcutting or grinding may be required in addition to abrasive blast cleaning of joints. Cleaning joints below the existing damaged waterstop and repairing the existing joint damage shall be considered to be specified extra work. Cost of repair of damage caused by the contractors operations shall be borne by the contractor. Getting a satisfactory joint may require the repairs of spalls, cracks, and expansions dams, and this is usually classed as other work. Supplemental funds should have been provided for all above noted extra work.

C. Saw Cutting

1. Type "A" and "AL" Seals

Joints to be sealed with type "A" Seals are to be saw cut to the dimensions shown on the contract plans. If for some reason the saw cut width has to be increased slightly to maintain a uniform groove width or to expose good sound concrete, it is essential to maintain a 1 to 3 depth to width ratio of the polyurethane seal.

Joints sealed under rehabilitation contracts with type "A" (modified) seals, shall have a groove width $\geq$ one inch and $\leq$ 1.75 inches. Joint seal depth shall equal 1/3 the joint width but must be $\geq$ 1/2 inch, (see Attachment No. 9).

The 1 to 3 depth to width ratio does not apply to the type "AL" seal. (Saw cut not required).

2. Type "B" Seals

In new construction, type "B" seals are to be saw cut as follows:

Joint movement calculation sheets, which include saw cut information, will be furnished by Design upon the request of the Structure Representative, when they are not included in the R.E. Pending File. (Attachment #2 is an example of a completed Joint Movement Calculation Sheet.)

Saw cutting shall not be started until the Type "B" seal material has been tested and released. The Transportation Laboratory will furnish each job with a copy of the test report showing the M.R. (Movement Rating) of the Type "B" seal groove width limits, (W_1 & W_2) which are necessary to determine the saw cut widths. The M.R. of the Type "B" seal must be equal to or greater than that shown on the contract plans.

The minimum saw cut (groove) depth is to be checked by cutting, a l/2" to 1" section of the actual seal to be used and placing it between two flat surfaces, such as 1"x4"x8", e.g. Place the top of the seal to the dimensions shown on the contract standard plan and compress it to the W_2 position. At this position determine the saw cut depth required per the standard plans.

At the time saw cutting is to begin, determine the groove or saw cut width as described on the joint movement calculation sheet shown in the example (Attachment No. 2). Mark and check the initial saw cut so that it can be used later to check the tolerance of the completed joint. This is very important because the joints are usually moving while the saw cutting is in operation. It is the Contractor's responsibility to adjust the cut accordingly to match the initial saw cut width and maintain the tolerances specified for the completed joint.

In new construction projects joint geometry is readily controllable, i.e. the size of the saw cut is set to accommodate the joint seal. Rehabilitation projects differ from new construction projects in that the width and condition of the joints require special consideration. The new joint seal must provide the required movement rating and also must be of sufficient size to fit the existing joint after saw cutting.

Rehabilitation projects require that both the Minimum W_1 (the maximum joint width at minimum temperature, after prestress shortening), and the M.R. be indicated on the plans. To ensure a correct fit, the W_1 of the joint seal must be greater than the minimum W_1 of the joint.

The Special Provisions require that the joint size be verified prior to ordering the seals. A joint should be remeasured only after that joint and its adjacent joints have been cleaned. Record the concrete temperature at the time of measurement.

Calculate the minimum W, required for the joints using the actual measurements.

$$\text{Min } W_1 = W_e + l/2 + \frac{(T_{str} - T_{min})}{1} \quad (2) \quad \frac{(\ 4\ )}{(100)}$$

Where:

Min W_1 = Maximum joint width in inches
W_e = Existing joint width in inches (measured at the widest point)

l/2 = Minimum practical concrete removal (l/4 inch each side of the joint)

T_{str} = Structure temperature, deg F (measured at the time the existing joint width was measured, W_e)

T_{min} = Minimum temperature at structure site - from form DS-D129

1 = Temperature range at structure site - from form DS-D129

2 = Thermal movement in inches/100 feet - from form DS-D129

4 = Contributary length in feet - from form DS-D129

Compare these. recalculated W_1's with the minimum W_1's shown on the plans. If they agree within 0.1 inch, the data shown on the plans does not need to be revised. If the new W_1's do not agree with the values shown on the plans, prepare a contract change order to revise the W_1's and state whether or not the movement ratings have changed.

If a calculated W_1 exceeds 4.25 inches, a compression seal should not be used. Contact the chairman of the Joint Seal Committee for a recommended course of action to follow.

Again, saw cutting should not start until test data for the seal to be used is available. Saw cut widths should be set to provide the minimum joint width possible. Due to the variables involved, saw cut widths should be calculated using the formulas given below and the narrower width chosen, provided it will work.

$$s_1 = W_1 \text{ -- } \frac{(T_{str} - T_{min})}{1} (2) \left(\frac{4}{100}\right)$$

$$s_2 = W_2 + \frac{(T_{max} - T_{str})}{1} (2) \left(\frac{4}{100}\right)$$

$$S_3 = w_e + l/2 = \text{Minimum practical saw cut width}$$

Where:

S_1, S_2, S_3, = possible saw cut widths

W_l = W_1 taken from test report (R-29)

W_2 = W_2 taken from test report (R-29)

W_e = Existing joint width in inches (measured at widest point)

l/2 = minimum practical concrete removal (l/4 inch each side of joint)

T_{str} = Structure temperature, deg F (taken at the time of measurement of W_e)

T_{min} = Minimum temperature at structure site - from form DS-D129
1 = Temperature range at structure site - from form DS-D129
2 = Thermal movement in inches/100 feet - from form DS-D129
4 = Contributary length in feet - from form DS-D129.

d. Installation

1. Tvpe "A" and "AL" Seals:

Be thoroughly familiar with the contract specifications and details and enforce them.

It is essential that the polyethylene foam be placed at a uniform depth to preclude excessively thin or thick sections. There is a successful relationship between the cohesion and the adhesion of the polyurethane seal if the proper shape and dimensions shown on the Standard Plan are maintained. Cut templates out of plywood to check the surface depths of the polyethylene foam and the polyurethane.

Type A (modified) seals require placing the joint seal and rod stock 3 inches up into the curb or rail on the low side of the deck at the curb or rail joint that lines up with the deck joint.

2. Type "B" Seals: (Attachment #l gives the properties for some brands of Type "B" seals.)

Again be thoroughly familiar with the contract plans and specifications and enforce them.

Repair all spalls and grind chamfer in advance of installing the seal.

As a final check, prior to installation, it is recommended to use a thin section of joint seal material and use it to check the saw cut depth throughout the length of joint. Place the seal section in the planned position and check to see that the dimensions shown on the Standard Plan are maintained. Most joint seal failures result from improper saw cuts or from the seal being placed too near the deck surface.
Bend type "B" seals 6 inches up into the curb or barrier rail on the low side of the deck. If the curb or rail joints don't line up with the deck joint, an attempt must be made to abut the joint seal to the face of the curb or rail so that it will provide a water tight seal.

3. Joint Seal Assemblies:

Details of a joint seal assembly are shown on the contract plans. The Structure Representative is to calculate the installation width of the joint seal assembly. Calculations are to be shown on the "Joint Movements Calculations" (DS-D129) sheet using a W_2 equal to l/2 inch minimum at maximum temperature.

The Special Provisions permit alternate joint seal assemblies which the Contractor may use in lieu of the joint seal assembly detailed on the Contract Plans.

If the Contractor proposes to use an alternate joint seal assembly, the Structure Representative shall send two copies of the initially submitted working drawings to Structures Design for a determination as to the adequacy of the proposed alternate joint seal assembly. When submitting the working drawings, point out that they detail a contractor proposed alternate joint seal assembly, and that they are submitted for an informal review by the Joint Seal Committee and by Structures Design.

If an alternate joint seal assembly is incorporated in the contract work, the Structure Representative should make the necessary changes on the "As Built" plans to indicate the details of the alternate joint seal assembly. An additional sheet may be necessary to show the "As Built" details. Do not submit the shop plans as "As Built" plans.

Note that prestressed concrete structures are expected to initially shorten about 0.50 in./100 ft. due to stressing. The total long-term shortening is anticipated to be 1.00 in./lOO ft. for post-tensioned bridges and somewhat less for pretensioned bridges. The difference between the long-term shortening (1.00 in.) and the initial shortening is equal to 0.5 in./100 ft. This is the value shown on the "Joint Movements Ca1culations" form (DS-D129) as "Anticipated Shortening for Post Tensioned Concrete Structures". For unusual situations when a substantial amount of time has elapsed between stressing and the placement of joint seals, an estimate may be made of the amount of prestress shortening that has occurred. Refer to Attachment No. 10 for an example,

4. Modular Joint Seal Assemblies (MR over 4")

Refer to the Special Provisions for details concerning the installation of modular joint seal assemblies. Any questions can be directed to your area Senior or the Joint Seal Committee.

5. Open Joint and Experimental Test Seals

Obtain the necessary brochures on installation procedures from your Construction Senior or the Chairman of the Joint Seal Committee if they are not included in the R.E. Pending File.

The proper installation width of open joints or experimental joint seals will be calculated from the Joint Movement Calculation Sheet. Determine the minimum width at maximum temperature (W_2) and insert this in Column 5. The adjustment of the width for temperature at time of installation will be the same as for the Type B Seal.

e. Expansion Joint Scribes

Scribes are to be placed at all expansion joints as shown on the attached instruction sheet (Attachment No. 3). Placement of the scribes at a location other than that shown may be required when special barrier rails are used. Use the 8" steel railing scribe, 3/4"x8" 24 gauge (Item No. 6635 1760 5) and 4" aluminum scribe plate, 1 1/2"x4" 16 gauge (Item No. 6635 1790 8) for joints having a movement rating of 2" or less. Use the 10" steel railing scribe, 3/4"x10" 24 gauge (Item No. 6635 1780 7) and 6" aluminum scribe plate, 1 1/2"x6" 16 gauge (Item No. 6635 1770 6) for joints having a movement rating greater than 2" Use 681-80-44 Rapid Set Epoxy (Item No. 8040 0100 4) to attach the scribes and plates to the rail. Scribes, plates and epoxy should be obtained from the District through the Resident Engineer. Order one scribe per expansion joint and epoxy at the rate of 1 unit (l/4 pint can of "A" and l/4 pint can of "B") per 20 scribe units. Skewed, or extra wide structures may require a scribe unit on the joint on both sides of the structure.

APPROXIMATE PROPERTIES
FOR
PREFORMED ELASTOMERIC JOINT SEALS
TYPE B_1

Manufacturer's Nominal Properties for Design Data Only
(See Note 4)

Catalog Number Depth (See Note 1)		Uncompressed W_0 (See Note 3)	Size D_0	Approx. M.R. (See Note 2)	W_1 Max. Groove Width	W_2 Min. Groove Width	Recommended Saw Cut
Brown	H-2503	2.5"	2.625"	1"	2.13"	1.13"	4.0"
Brown	H-3000	3.0"	3.25"	1"	2.55"	1.55"	5.0"
Brown	H-3500	3.5"	3.75"	1.5"	2.98"	1.48"	5.85"
Brown	H-4000	4.0″	4.25"	1.5"	3.40"	1.90"	6.0"
Brown	H-5000	5.0"	5.00"	2"	4.25"	2.25"	7.75"
Brown	H-6000	6.0"	5.5"	2.5"	5.10"	2.60"	9.25"
W.B.	WA-250	2.5"	2.75"	1"	2.13"	1.13"	3.56"
W.B.	WA-300	3.0"	3.38"	1"	2.55"	1.55"	4.31"
W.B.	WA-350	3.5"	3.5"	1"	2.98"	1.98"	4.44"
W.B.	WA-400	4.0"	4.38"	1.5"	3.40"	1.90"	5.00"
W.B.	WA-500	5.0"	5.0"	2"	4.25"	2.25"	5.94"
W.B.	WA-600	6.0"	6.0"	2.5"	5.10"	2.60"	7.75"

*W.B. - Watson Bowman

Notes:

(1) Brand Names other than those listed may be available.

(2) The actual Movement Rating equals (W_1-W_2). W_1 shall be the smaller of the values determined as follows:

1. 0.85 times the manufacturer's designated minimum uncompressed width of the seal (W_0).

2. The width of seal on the third successive test cycle of the pressure-deflection test, when compressed to an average pressure of 3.0 pounds per square inch.

W_2 shall be the width of seal determined on the third successive test cycle of the pressure-deflection test, when compressed to an average pressure of 4 times the pressure measured at the seal width W_1.

(3) Data shown may change significantly due to variations in extrusions. Dimensions must be verified in the field.

(4) Do not use these properties in lieu of actual test results. This is for additional information only. Actual values for W_1, W_2, and M.R. are obtained from test results performed by the Transportation Laboratory on the Report of Inspection of Material (Form TL-29).

DEPARTME[illegible] TRANSPORTATION
JOINT MO[illegible]MENTS CALCULATIONS
DS-D129

ROUTE :

1 DESIGNE F. C. Boyd
2 SPECIFICATIONS A. J. Pugh
3 JOINT SEAL COMMITTEE CHAIRMAN C. W. Jones
4 R.E. PENDING FILE S. Matney

AFTER AWARD OF CO[illegible]CT
5 R.E OR BR: COMPLETE & RETURN TO DIVISION OF STRUCTURES WITH JT SEAL REPORT.
6 JOINT SEAL COMMITTEE CHAIRMAN
7 MAINTENANCE

EXPENSE AUTHORIZATION	DIST	COUNTY	ROUTE	PM	BRIDGE NAME & NO.
03-000000	03	Sac	5	0.2/2.4	Dry Creek O.C. 29-000

TYPE STRUCTURE: Reinf. Conc. Box & CIPP/S
TYPE ABUTMENT: A1 - 70T Piles / A7 - Spd. Ftg.
TYPE EXPANSION (2" ELASTO PADS ETC.): Bent 3 - Steel Hangers; Bent 5 - 2½" Pads

TEMPERATURE EXTREMES (from preliminary report) MAX 110 °F
− MIN 23 °F
TYPE OF STRUCTURE = RANGE 87 °F ①

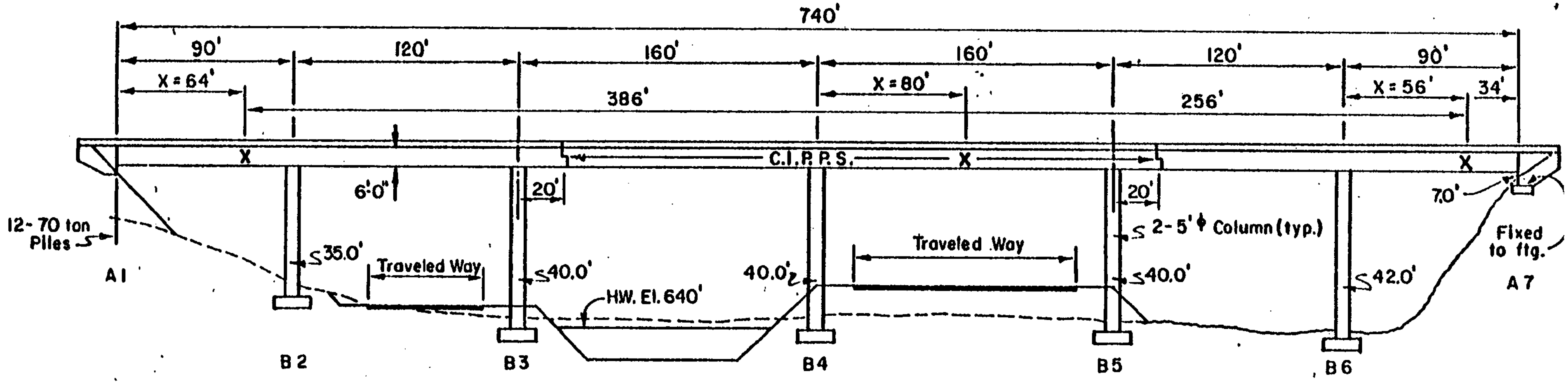
TYPE OF STRUCTURE		② THERMAL MOVEMENT (in. /100 ft)	+	ANTICIPATED SHORTENING (in. / 100 ft)	=	③ MOVEMENT FACTOR (in. / 100 ft)
STEEL	" (°F) (.0000065)(1200) =		+	0.00	=	
CONCRETE (Conventional)	" (87 °F) (.0000053)(1200) =	0.55	+	0.06	=	0.61
CONCRETE (Pretensioned)	" (°F) (.0000053)(1200) =		+	0.12	=	
CONCRETE (Post tensioned)	" (87 °F) (.0000053)(1200) =	0.55	+	0.50	=	1.05

ⓐ TO BE FILLED IN BY DESIGNER: F. C. Boyd DATE 9-1-74
TO BE FILLED IN BY BR: W. T. Trustworthy DATE 7-4-76 & 7-5-76

LOCATION	SKEW ○ DO NOT USE IN CALC	④ CONTRIB LENGTH (Ft)	CALCULATED MOVEMENT (Inches) ③ × ④/100	M.R. (Inches) ROUND UP TO ½"	SEAL TYPE A, B, (OTHERS) OR OPEN JOINT	ⓑ SEAL WIDTH LIMITS: CATALOG NUMBER	W1 (Inches) MAX.	⑤ W2 (Inches) MIN. MAX TEMP	ⓒ GROOVE (SAW CUT) WIDTH OR INSTALLATION WIDTH: STRUCTURE TEMP. (°F) *	⑥ ADJUST FROM MAX TEMP (Inches) Δ°/① × ② × ④/100	INSTALLATION OR GROOVE WIDTH AT TEMP. LISTED (Inches) W=⑤+⑥
Abut. 1 (Conv.)	0	64	0.39	½	A	PRC 3105					1.50
Span 3 Hinge (Conv.)	0	166	1.01								
Span 3 Hinge (CIPP/S)	0	220	2.31								
Span 3 Hinge Total	0	386	3.32	3½	Joint Seal Assembly	Arm. Neoprene	4.50	0.50**	90	0.49	0.99
Span 5 Hinge (CIPP/S)	0	100	1.05								
Span 5 Hinge (Conv.)	0	156	0.95								
Span 5 Hinge Total	0	256	2.00	2	Type B	Brown H-5000	4.25	2.00	94	0.26	2.26
Abut. 7 (Conv.)	0	34	0.21	½	A	PRC 3105					1.50

ⓐ SHOW LINE DRAWING OF STRUCTURE ON REVERSE SIDE, SHOW POINTS OF NO MOVEMENT AND CONTRIBUTORY LENGTHS RETAIN COPY FOR DESIGN CALCULATIONS FILE.
ⓑ INFORMATION FROM TRANSPORTATION LAB REPORTS ⓒ GROOVE WIDTH ADJUSTMENT BASED ON Δ° = (MAX TEMP EXTREME) MINUS (SUPERSTRUCTURE TEMPERATURE).
* MEASURE SUPERSTRUCTURE TEMPERATURE BY PLACING BULB OF CONCRETE THERMOMETER 6" ± INTO EXPANSION JOINT.
** If alternative assembly is used, select correct W2 from sketches shown on Attachment No. [illegible]

CALCULATION OF POINTS OF NO MOVEMENT



	A1	B2	B3		B4	B5		B6	A7	
$I\ (Ft)^4$	1.38	61.36	61.36		61.36	61.36		61.36	102	
L (Ft)	5.50	35.0	40.0	Σ	40.0	40.0	Σ	42.0	7.0	Σ
P (kips) @ 1" side sway	1200	618	415	2,233	415	415	830	359	Will slide 600 ~~130,000~~	95[illegible]
D (dist. from 1st. member of frame)	0	90	210		0	160		0	90	
P x D / 100	0	556	870	1,426	0	664	664	0	540	54[illegible]

$X = \frac{\Sigma P \times D/100}{\Sigma P} (100') = \frac{1426}{2233} (100) = \underline{64'}$

$\frac{664}{830} (100) = 80'$

$\frac{540}{959} (100) = \underline{56'}$

Notes:

Width Str. = 78'
Dia. Col. = 5'0"
K / Pile @ 1" defl. = 100
X = Point of No Movement

Assumptions:

1. Super str. Inf. rigid
2. Col's fixed top & bottom
3. Abut. diaph. will slide @ a force equal to D.W.
4. E (piles) = 4×10^6
   E (columns) = 3×10^6

$P\ (Col.) = 12\ EI \frac{\Delta}{L^3}$

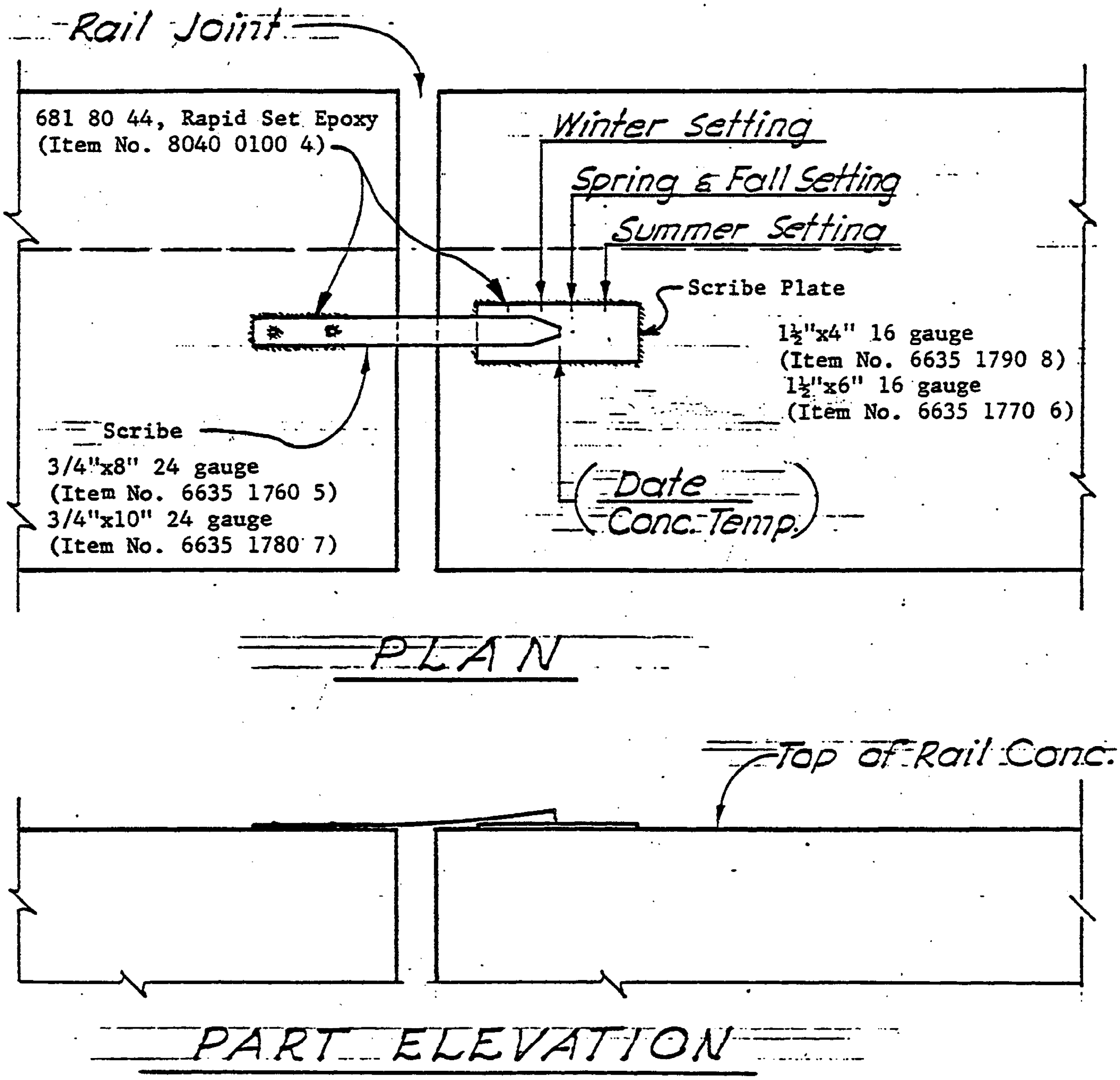
@ 1" defl. $= \frac{432\ I}{(\frac{L}{10})^3}$

D.W. Abut 5 = 600K (assume linear up to 1" defl.)

$I\ (abut) \approx \frac{78}{12} (2.5)^3$

≈ 102

EXPANSION JOINT SCRIBE



NOTES:

1. Install one scribe at each deck joint on the most convenient side of the roadway i.e., widest shoulder. Use 8" scribe and 4" plate for joints having movement rating of 2" or less. Use 10" scribe and 6" plate for joints having movement rating greater than 2".
2. Place scribe on top of the concrete portion of the barrier railing.
3. Sand or wire brush surfaces of scribe and concrete to insure good adhesion.
4. Mix only enough epoxy for one scribe and plate when using the 681-80-44 Rapid Setting Epoxy. (5 min, pot life @ 70°)
5. Use weight on a piece of paper to hold the scribe down on the concrete surface while the epoxy is setting.
6. Mark the Initial Position of the scribe, date, and concrete temperature on the plate as shown with a scriber. Measure the concrete temperature by placing the bulb of a concrete thermometer 6" + into the deck section, if possible, or at any convenient location to obtain the approximate superstructure temperature.

SAMPLING AND TESTING OF TYPE "B" JOINT SEALS

The following revised instructions for sampling and testing Type "B" joint seals have been issued by the Transportation Laboratory. The procedures are currently in use. If there are any questions call Richard Spring at (916)739-2314.

1. Following the manufacturing of a given quantity of various sizes (Movement Ratings) of joint seal materials for use on Caltrans contracts, such as:

   MR=1" (1500 LF)
   MR=1 1/2" (1000 LF)
   MR=2" (1000 LF)

   The manufacturer will notify our Caltrans Laboratory (Richard Spring (916)739-2314).

2. Mr. Spring will arrange for an independent inspection agency to contact the manufacturer for the purpose of sampling the various lots of materials at the source.

3. The sampling agency will obtain one 3' long sample of each size and lot of material for every 500 LF 2 and send to our laboratory for testing along with the manufacturer's test report. The manufacturers lot number will appear along the length of the seal.

4. Following satisfactory testing, the manufacturer will be notified and the material will be set aside for <u>stock</u> to be used on Caltrans contracts only.

5. As the manufacturer receives orders and makes shipments to the individual contracts, form letters will be sent to our Caltrans Laboratory and with the shipment to the jobsite. This letter will contain the following information:

   a. Name and address where the seal is being sent.

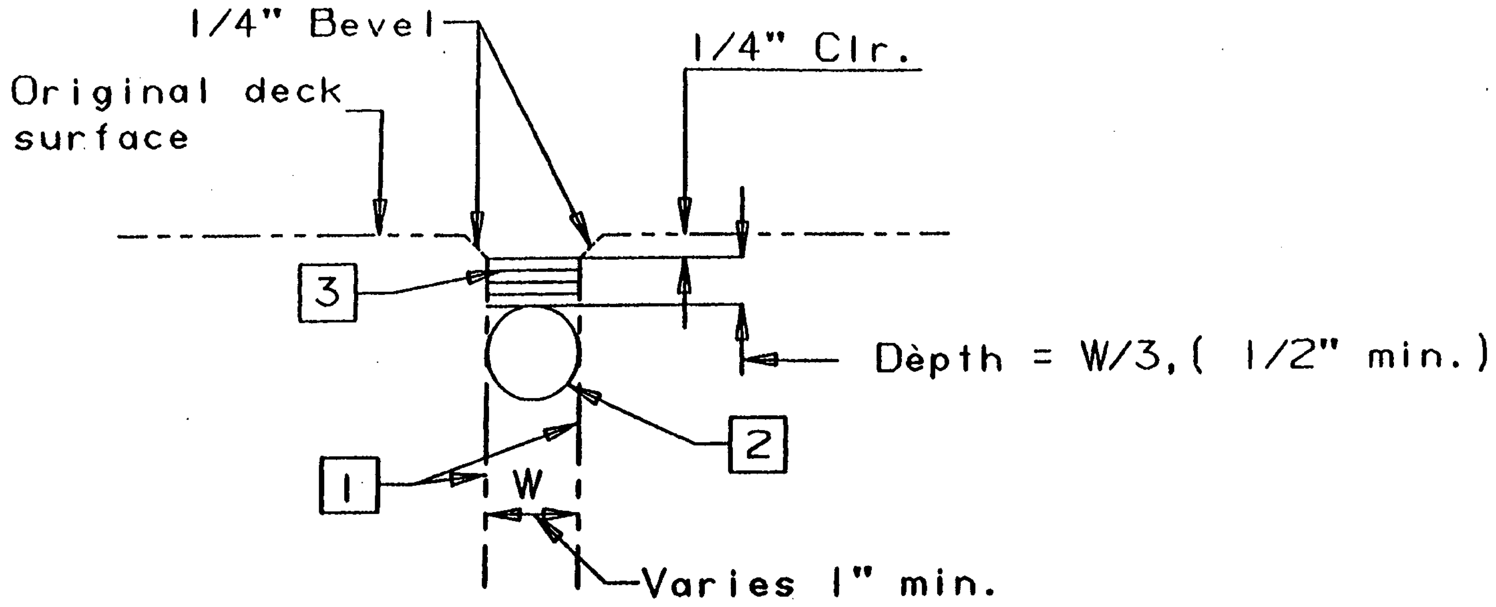
   b. State Contract Number.

   c. Size, quantity and movement rating of the seal.

   d. The Lot Number identifications.

   e. The TransLab's test number (SM number).

Upon receiving the letter from the supplier as to where the seal is being sent, the TransLab will send to the RE or Structure Rep a copy of the test report for the particular lot of material. Included on the test report will be the W_1 and W_2 values for the seal. The RE or Structure Rep should verify the lot number on the seal with the test report lot number.



JOINT SEAL TYPE A MODIFIED (MR 1/2")

No Scale

Notes:

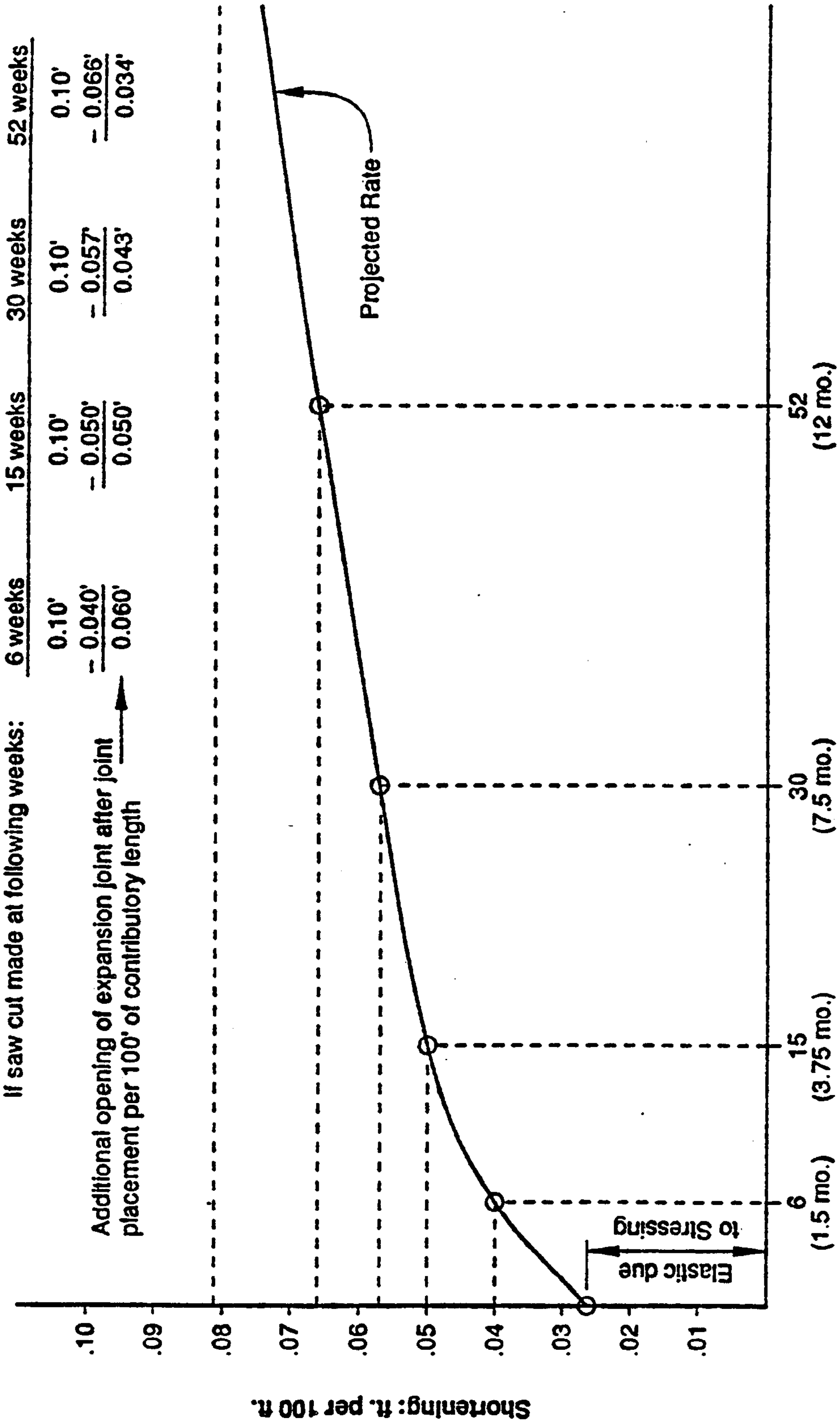
1. If required, sawcut or grind transverse joints to the minimum (W) shown. Clean and abrasive blast joint.
2. Install commercial quality closed cell polyethylene rod stock with glazed surface. Diameter = joint width + 1/4".
3. Install joint seal. Place joint seal 3" up into curb or rail on low side of deck.

For details not shown see B6-21

Prestress Shortening

Assume long term total shortening is 0.10'/100'

If saw cut made at following weeks:	6 weeks	15 weeks	30 weeks	52 weeks
	0.10'	0.10'	0.10'	0.10'
	– 0.040'	– 0.050'	– 0.057'	– 0.066'
Additional opening of expansion joint after joint placement per 100' of contributory length →	0.060'	0.050'	0.043'	0.034'



7

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 135-2.1** **MISCELLANEOUS CONSTRUCTION MATERIALS**
Approved: ______ **R. P. SOMMARIVA, Chief** **Division of Structure Construction**	**Date: January 20, 2000** **Expires: July 1, 2001** **Supersedes: None**

Subject: Release and Reporting of Values for Type B Joint Seal

The last paragraph of Sheet 2 of 7 of Bridge Construction Memo 135-2.0 is modified to read:

> Saw cutting shall not be started until the Type B seal material has been verified as having successfully been tested by the Division of Materials Engineering and Materials Testing Services (DMETS). The contractor/subcontractor will need to provide information to the Structure Representative regarding the manufacturer, lot number, date of manufacture and movement rating of the joint seal intended to be used prior to bringing the seal to the job site. The Structure Representative will verify the successful testing by contacting DMETS at 916.227.7263 and to obtain the W_1 and W_2 values for the lot of seal that will be used by the contractor. The movement rating (M.R.) ($W_1 – W_2$) of the Type B seal must be equal to or greater than that shown on the contract plans.

When contacting DMETS for W_1 and W_2 information, the Structure Representative (caller) should have the following information readily available:

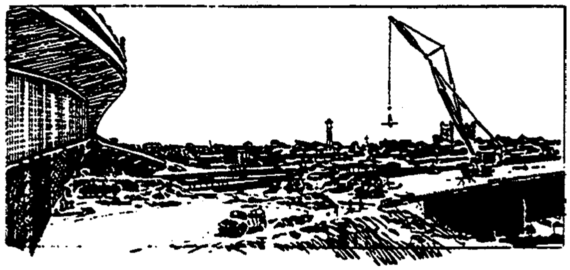
Manufacturer of the Type B Seal
Lot number shown on the side of the Type B Seal
Date of manufacture
Movement Rating for the seal

If requested, a copy of the DMETS test report for the Type B seal can be sent to the Structure Representative. It is important to note that the Type B Joint Seal will arrive at the jobsite without any state inspection release tags and no report of inspection document (TL-0029).

Background:

Type B joint seal is normally supplied by one of two sources from producers in the Midwest. As part of the manufacturing process, the various sizes of Type B Joint Seals are identified on the side of the seals with a Lot Number which represents a certain quantity. A sample of each lot of material is sent to DMETS in Sacramento for testing. Following the successful testing, the manufacturer is notified that the lot or lots of Type B Joint Seals are acceptable. At the time testing is performed, the lab does not know on which state contracts the Type B Seals will be used, therefore no information can be sent to the Structure Representative.

c: BCR&P Manual Holders
Consultant Firms
PStolarski, Chief Division of Materials Engineering and Testing Services
RWWolfe, Acting Chief Office of Structural Materials
BPieplow, Acting Construction Program Manager



COMPRESSIVE DEFLECTIONS OF FIBERGLASS OR STEEL REINFORCED ELASTOMERIC BEARING PADS

Elastomeric Bearing Pads consist of alternate layers of elastomer and steel sheet or fiberglass fabric reinforcement.

The compression deflections for steel and fiberglass reinforced elastomeric bearing pads can be reliably predicted within the normal range of construction tolerances. Transportation Laboratory tests have found that compressive strain is dependent upon two factors - compressive stress and shape factor. In addition, tests showed that compressive stress/strain behavior of fiberglass or steel reinforced pads is not significantly dependent upon overall pad thickness. In most situations the compressive deflection of the pad will be so small as to not effect the profile of the bridge. However, in the case of a hinge the magnitude of the deflection should be investigated, as it may be significant.

Attachment No. 1 of this Bridge Construction Memo shows two tables with families of curves which can be used to predict compressive deflection based on stress, shape factor, and strain. There is a separate table for fiberglass reinforced or steel reinforced pads which apply regardless of overall pad thickness. If long term compressive creep is to be included in the prediction, the strain values obtained from Attachment No. 1 should be increased by 25 percent.

Lab tests have shown that fiberglass and steel reinforced pads recover from dynamic creep caused by live loads. Therefore, dead load stress only will be considered in determining compressive deflection. Current bridge design practice limits the nominal compressive stress on a pad to 800 psi due to dead load and live load, not including impact. For steel reinforced pads with a shape factor $\geq$ 7.5, the average pressure shall not exceed 1000 psi, For calculating compressive deflection in the field, a dead load stress of 600 psi should be used. If a more accurate value of dead load stress is desired, contact the Structures Design Section responsible for your contract plans.

For special situations where extreme accuracy is desired, sample pads can be tested by the Transportation Lab to determine the stress/strain behavior of each lot of pads.

SAMPLE CALCULATION

Consider a 12"x18"x4" fiberglass reinforced bearing pad

Assume compressive stress of 600 psi

$$\text{Shape Factor} = \frac{\text{width x length}}{\text{width + length}} = \frac{12(18)}{12+18} = 7.20$$

From Attachment No. 1

Strain = 4.0% = .040

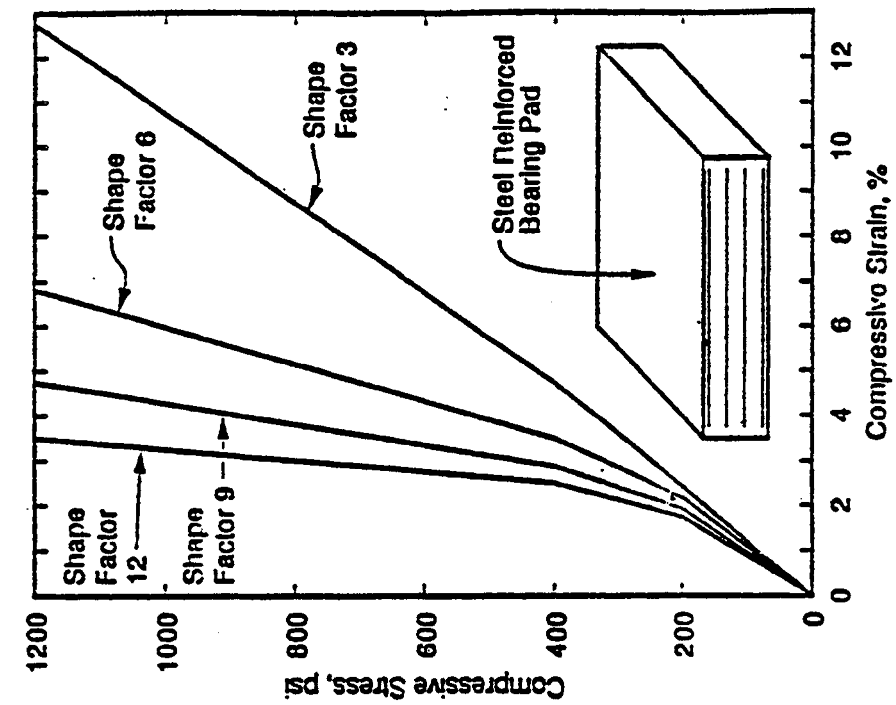
Compressive deflection including long term creep of 25% is equal to:

(Total Pad Thick.) (Strain) (125%)

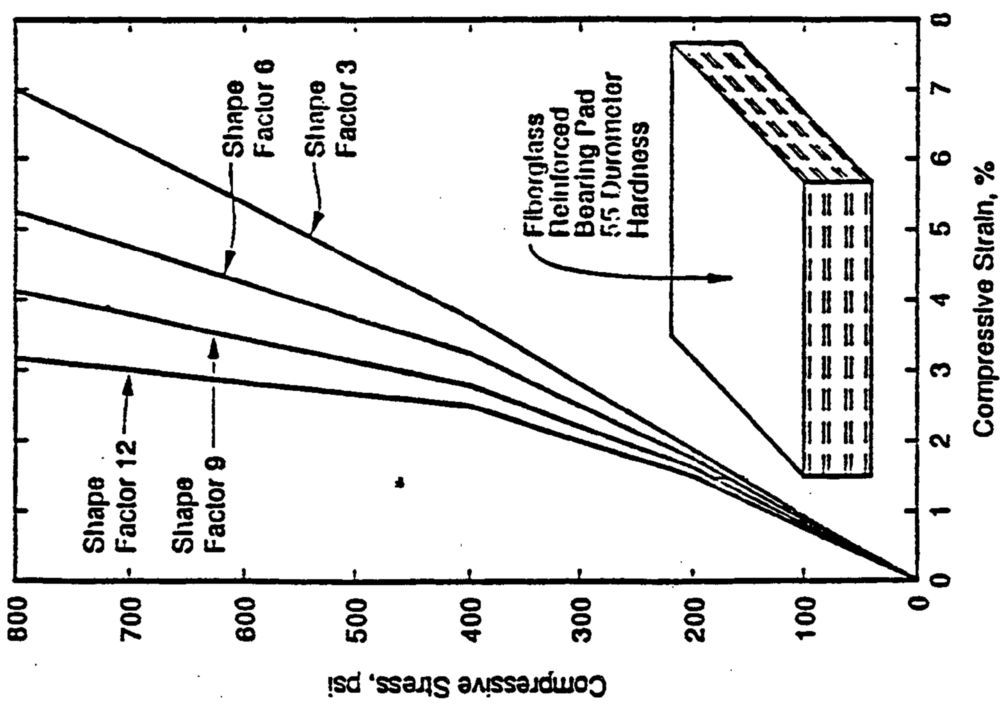
4.00 (.040) (1.25) = .20"

Say 3/16"

The bearing pad thickness shown on the plans will be that for fabric reinforced pads. Note that steel reinforced bearing pads are thicker than the corresponding fabric reinforced pads. If the thickness of a fabric reinforced pad is T (inches), the thickness of the corresponding steel reinforced bearing is 1.15 T. See Attachment No. 2 for a table of thicknesses for Steel Reinforced pads. For pads more than l/2 inch thick, it is the responsibility of the contractor to notify the Engineer in writing of the type of pad to be used. Bearing seat elevations must be set to correspond to the bearings to be used.



Shape Factor 3
Shape Factor 6
Shape Factor 12
Shape Factor 9
Steel Reinforced Bearing Pad
1200
1000
800
600
400
200
0
Compressive Stress, psi
0
2
4
6
8
10
12
Compressive Strain, %



Shape Factor 6
Shape Factor 3
Shape Factor 12
Shape Factor 9
Fiberglass Reinforced Bearing Pad 55 Durometer Hardness
800
700
600
500
400
300
200
100
0
Compressive Stress, psi
0
1
2
3
4
5
6
7
8
Compressive Strain, %

MEMO TO DESIGNERS:

Background

Our policy has been to standardize on l/2" layers of elastomer. Until recently, we used very thin steel plates and a minimal elastomer cover at the top and bottom for the steel reinforced pads. The minimal thickness of cover and of steel, was ignored and the bearing-thickness shown on the plans was the sum of the l/2-inch thick layers. This resulted in a simple, standard Caltrans procedure for the design and manufacture of both the fabric reinforced and steel reinforced bearing pads.

The steel reinforcement option was removed from the 1981 Standard Specifications because the bearing manufacturers could not properly mold the bearing with the thin steel plates.

Current Policy

The current specifications for elastomeric bearings permit the use of the steel reinforced bearing as an option. However, the proper design of the steel reinforced bearings requires 14 gauge (0.075 inch) steel plates, full l/2-inch elastomer layers between the plates and a l/4-inch cover top and bottom. Therefore, the steel reinforced bearing will always be thicker than the corresponding fabric reinforced (fiberglass) bearing pad.

Design

There is no change in the design procedure. The designer will continue to design for the required number of l/2-inch layers and call out the thickness of the bearing as the sum of the l/2-inch thick layers. In permitting the use of the steel reinforced bearing as an option, the specifications require that the contractor notify the Resident Engineer of their choice. If the steel reinforced bearing is selected, the bearing seat elevation will be adjusted (lowered) by the Resident Engineer to allow for the increased thickness. The minor increase in compression on the steel plates due to the l/8" side cover may be ignored.

For most cast-in-place concrete, precast concrete and steel superstructures, there should be no difficulty in adjusting the bearing seat elevation at the time the contractor selects the bearing type. In general, there is no need for the designer to be concerned with the choice.

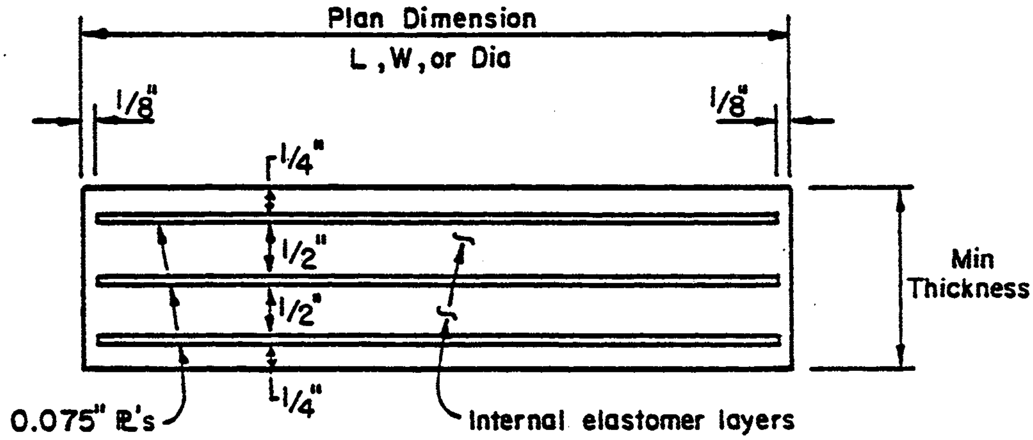
For some applications, the designer may want to limit the bearings to only one of the two types. If this is done, indicate the type selected on the plans as follows:

"Elastomeric Bearing Pads (fabric reinforced)" or,

"Elastomeric Bearing Pads (steel reinforced)"

Steel reinforced bearings are recommended where anchor bolt holes are required through the bearings. To assist the designer, the total thickness for steel reinforced bearings is tabulated below:

Design Thickness	Number of l/2" Layers	Number of Steel Plates	Actual Thickness Minumum	Maximum
1″	2	2	1.15	1.29
1.5″	3	3	1.73	1.89
2"	4	4	2.30	2.48
2.5″	5	5	2.88	3.08
3″	6	6	3.45	3.67
3.5″	7	7	4.03	4.27
4.0"	8	8	4.60	4.86



STEEL LAMINATED ELASTOMERIC BEARING

BRIDGE CONSTRUCTION MEMO 135-5.0

MISCELLANEOUS CONSTRUCTION MATERIALS

April 11, 2003

Sheet 1 of 2

Volume II

MECHANICAL ANCHORAGE DEVICES

Expansion anchorage devices seldom develop the full tensile strength of a stud or a bolt, and are therefore less desirable than cast-in-place bolts and inserts. For this reason, expansion anchorage devices are generally used only for attaching fixtures such as signs, ladders, utilities, and temporary railings to hardened concrete.

Inspections by Structures Maintenance Engineers have disclosed instances of loose anchorage devices for bridge-mounted signs. The lack of proper anchorage had gone undected because a headed bolt was used instead of a threaded stud. When the bolt was tightened against the fixture mounting plate, it pulled the anchorage loose from the concrete. The anchorage then pressed against the other side of the plate, and further tightening gave the impression that the fixture was securely attached when actually it was loose.

In order to minimize this problem, plans call for a threaded stud instead of a headed bolt. The Standard Specifications require that the expansion anchor be recessed l/2" to 1" below the concrete surface after it has been expanded. This allows the inspector to observe if the anchorage has been seated initially, and if it is properly holding at the time the fixture is attached. The plans also call out the diameter of the stud. Galvanizing requirements are given in the specifications.

In the event that the aforementioned details are not shown on the project plans, the Structure Representative should insist that studs and nuts be used instead of bolts. Note that the stud diameter should be 5/8" if the plans call for a 5/8" anchorage device. The other aspects of the expansion anchorage shall conform to the project plans and specifications.

Note that when anchorages are expanded by driving the expansion element over an expander plug, a sufficient thickness of concrete must be provided behind the plug to resist the driving force. The drilled hole for the anchorage must also be true to size and shape so to assure the fullest bearing of the expanded anchor against the concrete.

All concrete anchorage devices shall be subject to the approval of the Engineer. Current approval lists can be found at http://www.dot.ca.gov/hq/esc/approved_products_list/. On the page are ‘Cartridge Epoxies’ and ‘Mechanical Expansion Anchors’ that link to the latest approved products. If the proposed MEA does not appear on the working list, approval shall be contingent upon the submittal to the Engineer of sample concrete anchorage devices, manufacturer's instructions, and certified results of tests indicating compliance with specification requirements.

In summary, the Structures Representative should be sure that all expansion anchorage installations conform to the following:

1. Be sure the anchorage device is listed on the approved working list (Website given).

2. Proper size hole is drilled.

3. Use threaded studs and not headed bolts.

4. Use galvanized studs. (Not black steel)

5. Be sure the expansion part of the anchorage is properly recessed below the surface after it has been expanded.

6. Never accept a stud of smaller diameter than the stud or anchorage device size called for on the plans.

Another useful resource can be found in the Bridge Design Aids. Pages 81 through 92 of Chapter 5, Concrete Design, show a properly installed anchorage and indicates where anchors may be used and what will be shown on contract plans. The complete manual is available on Structure Design's website at the following address:
http://onramp.dot.ca.gov/hq/esc/sdsee/design_technical_services/publications/bridge_design_aids.shtml

Volume II

BRIDGE CONSTRUCTION MEMO 145-0.0

MISCELLANEOUS INFORMATION

July 1, 2001

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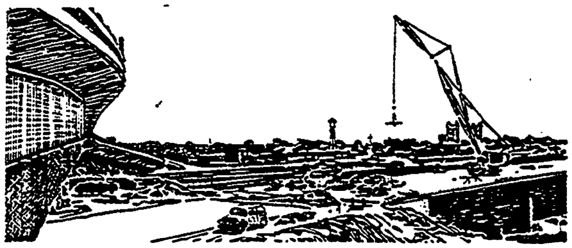
145-10.0	7-1-01	TIEBACK WALL CONSTRUCTION CHECKLIST

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction



BRIDGE CONSTRUCTION MEMO 145-1.0

MISCELLANEOUS INFORMATION

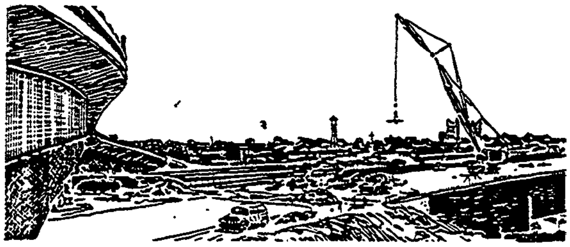
May 4, 1981

Volume **II**

HINGE EQUALIZING BOLTS

"Equalizing bolts* must be installed at the hinges of box girders and "T"-beam structures which have more than one hinge, even though there are "restrainer units" detailed to be installed at the hinges.

The "equalizing bolts" are needed in addition to the "restrainer units" to assure that hinge movements are equalized before the normal limits of the elastomeric pads are exceeded.

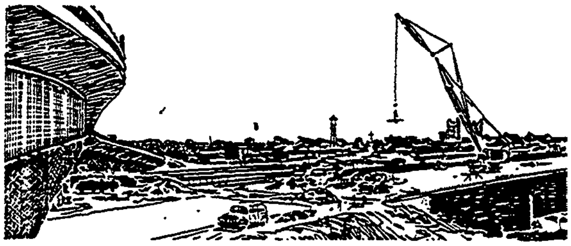


SAMPLING AGGREGATE FOR ASPHALT CONCRETE

Structure Construction personnel are occasionally placed in responsible charge of contracts having work which includes asphalt concrete paving or overlays.

On contracts having asphalt concrete paving or overlays, it is necessary to provide a representative sample of the aggregate to be used at least 30 days, but not more than 60 days, prior to paving. This sample is needed to determine the preliminary asphalt concrete mix design. The sample size will be designated by the Engineer.

Refer to Section 6-39 Asphalt Concrete and Section 8-01 Sample Types and Frequencies of the CONSTRUCTION MANUAL for detailed instructions. Communicate with the District coordinator or plant specialist for information on local asphalt plants.



BRIDGE CONSTRUCTION MEMO 145-3.0

MISCELLANEOUS INFORMATION

May 4, 1981

Volume II

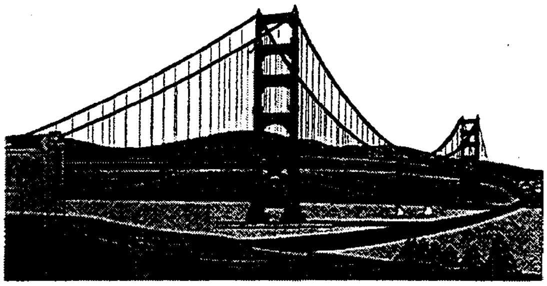
SUPPORT SYSTEMS FOR PORTIONS OF PERMANENT BRIDGES WHICH ARE TEMPORARILY UNSTABLE

Occasionally portions of permanent bridges are unstable during some stages of construction. Examples of such unstable portions of bridges are sloping abutments, and bent columns, where these components are hinged at the footing and not yet stabilized by completion of the superstructure. Numerous other conditions of instability of portions of the permanent structure may occur during individual phases of construction.

It is essential that the Structure Representative determine if, when, and where such conditions of instability may occur. When it is determined that a portion of the permanent structure will be unstable, the Contractor should be required to submit working drawings showing details of his proposed temporary support system. (Section 5-1.02 of Standard Specifications) The Structure Representative shall review these working drawings to ascertain that the proposed support system is adequate to provide the necessary stability. It is especially important that these procedures be followed when there is an unstable portion of a bridge adjacent to a railroad or to an area occupied by public traffic.

Contractors frequently make use of wire rope "guys" to temporarily support unstable portions of bridges. The Structure Representative should be alert to the fact that a poorly designed wire rope "guy" system, or the improper installation of wire rope "guys", may result in a catastrophic failure of that portion of permanent structure that is being stabilized by the "guy" system.

For information concerning the proper use of wire rope, refer to the report prepared by John MacNeill entitled "The Use of Wire Rope Guys and Restrainers for Concrete Forms and Structural Components", dated May, 1975. This report is available to all registered Civil Engineers of the Office of Structure Construction.



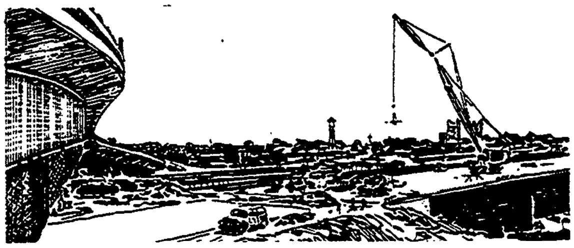
CONTRACTOR DESIGNED TEMPORARY BRIDGES OR OTHER TEMPORARY FACILITIES

If the Contractor proposes to construct a temporary bridge or other temporary facility across a traveled way being used by public traffic, the Contractor shall furnish the Structure Representative with working drawings and calculations detailing the location and design of the structure.

The Structure Representative will forward the aforementioned working drawings to the Sacramento Structure Construction Office for review by the Office of Structure Construction Falsework Section. The drawings should be forwarded with a letter describing loads to be supported, date Contractor wishes to begin construction, the design criteria used, and any other information pertinent to the review.

Temporary bridges or other temporary facilities not shown on the contract plans shall conform to Section 5-1.02, Plans and Working Drawings and Section 7-1.09, Public Safety, of the Standard Specifications.

Construction of the temporary bridge or other temporary facility should not be started until after the working drawings have been approved by the Engineer.



BONDING DOWELS IN DRILLED HOLES

There are three materials currently specified to bond a reinforcing dowel into a drilled hole in existing concrete.

Grout

Where plans or special provisions do not specify a bonding material a cement paste is to be used as outlined in Section 51-1.13 of the Standard Specifications. Special Provisions will contain a section "Drill and Grout Dowels where this is a separate contract item.

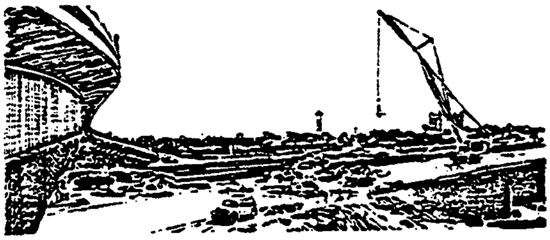
Magnesium Phosphate Concrete

Plans may indicate "Drill and Bond Dowel ... etc." Special Provisions will have a section "Drill and Bond Dowels" which refers to Section 83-2.02D(l) of the 1988 Standard Specifications. This specification is used where concrete barriers are to be constructed on existing bridge decks and can be used in other locations where shown on the plans. (Note - Corresponding 1984 Standard Specifications refer to Section 51-l.13 for bonding requirements.)

Epoxy

Plans in this case will indicate the use of epoxy bonding material. Special Provisions will have a section "Drill and Epoxy Bond Dowels" and there will be a separate contract item to cover the work.

The Structure Representative should not permit a substitution of the specified materials unless approved by the Structure designer.



BRIDGE CONSTRUCTION MEMO 145-6.0

MISCELLANEOUS INFORMATION

December 28, 1981

Volume II

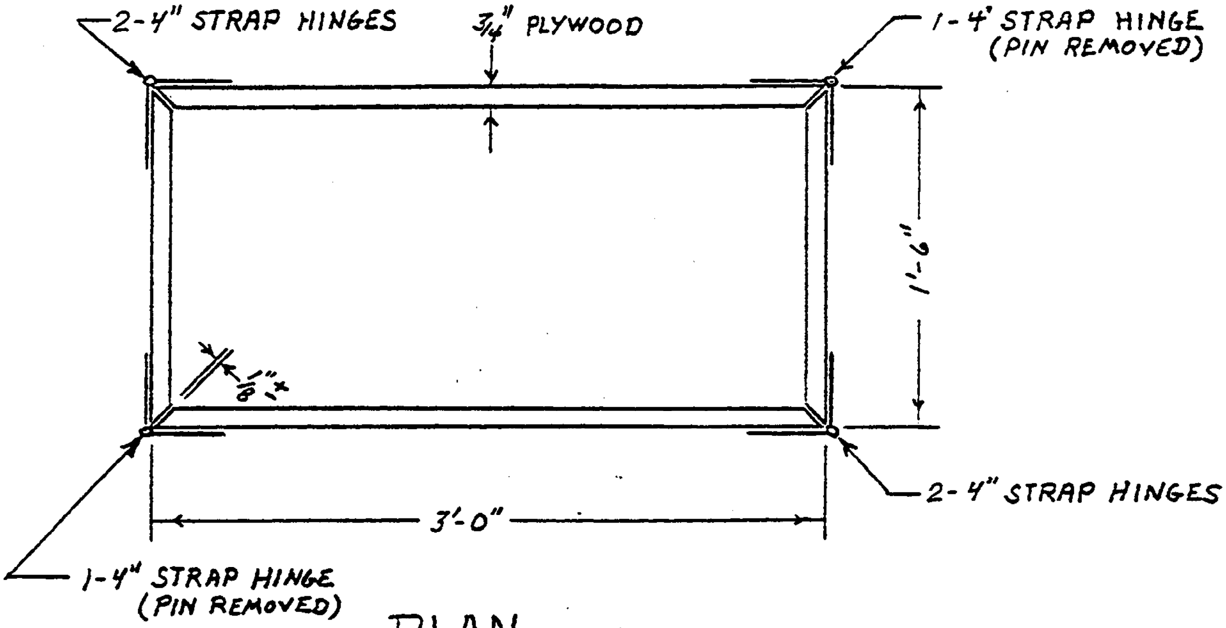
CONTAINER FOR USE IN PERFORMING THE BALL PENETRATION TEST

California Test 533 states that "The ball penetration test may be made on concrete in a wheelbarrow, buggy, or other container or after it has been deposited in the forms or on the subgrade." The test method requires that a minimum of three individual readings be taken for each penetration determination, and that the individual readings shall be at least nine inches between centers. Also, the minimum horizontal distance from the centerline of handle to the nearest edge of the level surface on which the test is made shall be six inches, and the depth of concrete above the bottom of the container or reinforcement shall be six inches for one inch maximum size aggregate and eight inches for larger size aggregate.

Attachment No. 1 to this Bridge Construction Memo is a drawing showing the details of a container which can be easily constructed and which will allow ball penetration tests to be made in conformance with California Test 533 requirements. As shown on the drawing, the container collapses and is therefore easy to store, transport, and handle. The container's size provides sufficient concrete to cast at least eight concrete test cylinders of the size required by California Test 540.

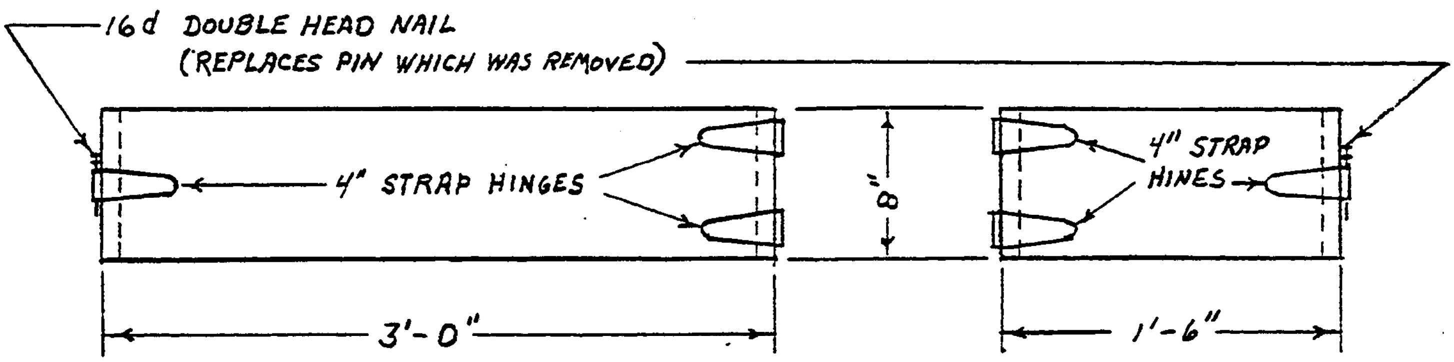
CONCRETE CONTAINER

FOR USE IN PERFORMING CALIFORNIA TEST METHOD 533



PLAN

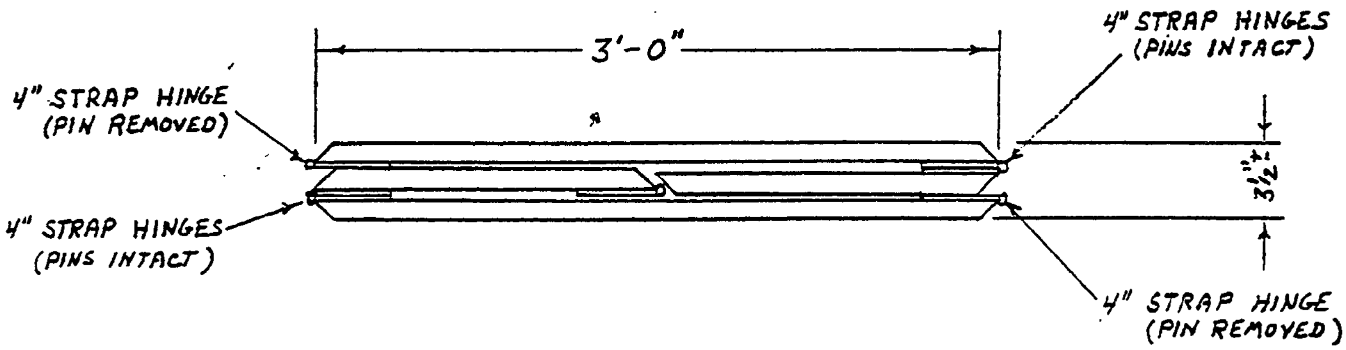
NOTE: FOR DURABILITY, THE CONTAINER SHOULD BE TREATED WITH THOMPSON WATER SEAL OR A SIMILAR TYPE SEAL.



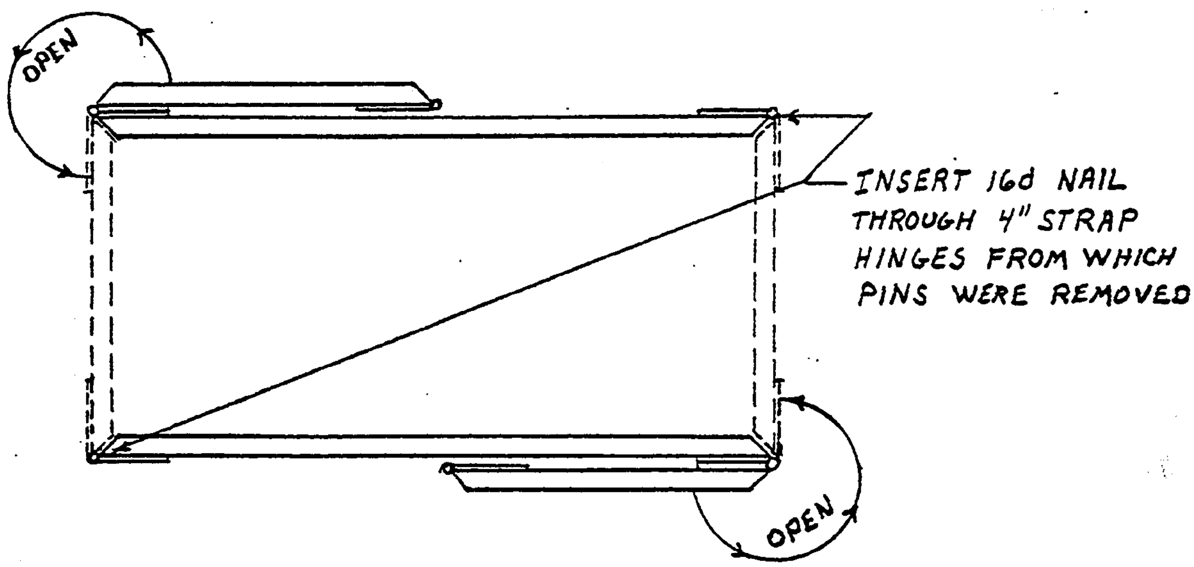
ELEVATION

END VIEW

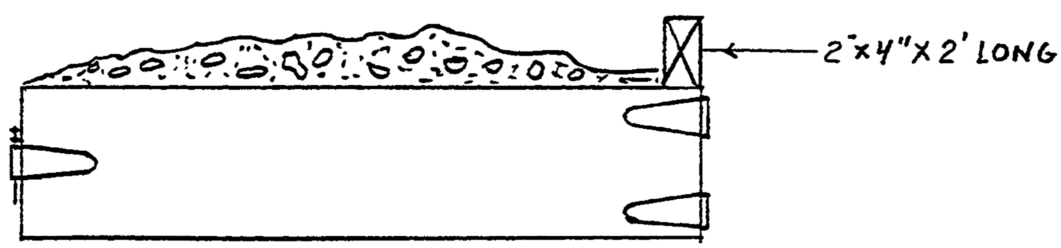
BRIDGE CONSTRUCTION MEMO 145-6.0
ATTACHMENT NO. 1 (12-28-81)
Sheet 1 of 2



CONCRETE CONTAINER IN COLLAPSED CONDITION

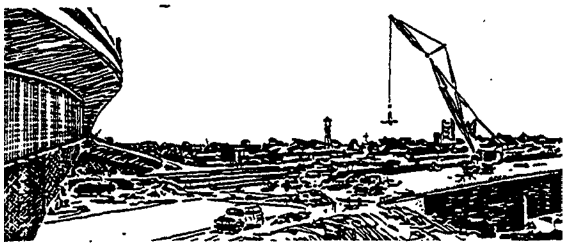


SETTING UP CONCRETE CONTAINER



CONTAINER FILLED WITH CONCRETE

NOTE: USE 2"x4" x 2' LONG TO STRIKE OFF CONCRETE LEVEL WITH TOP OF CONTAINER PRIOR TO PERFORMING CALIFORNIA TEST METHOD 533



ADJUSTABLE TEMPLATE FOR CHECKING PROFILES OF DUCTS IN POST - TENSIONED, PRESTRESSED, CONCRETE GIRDERS

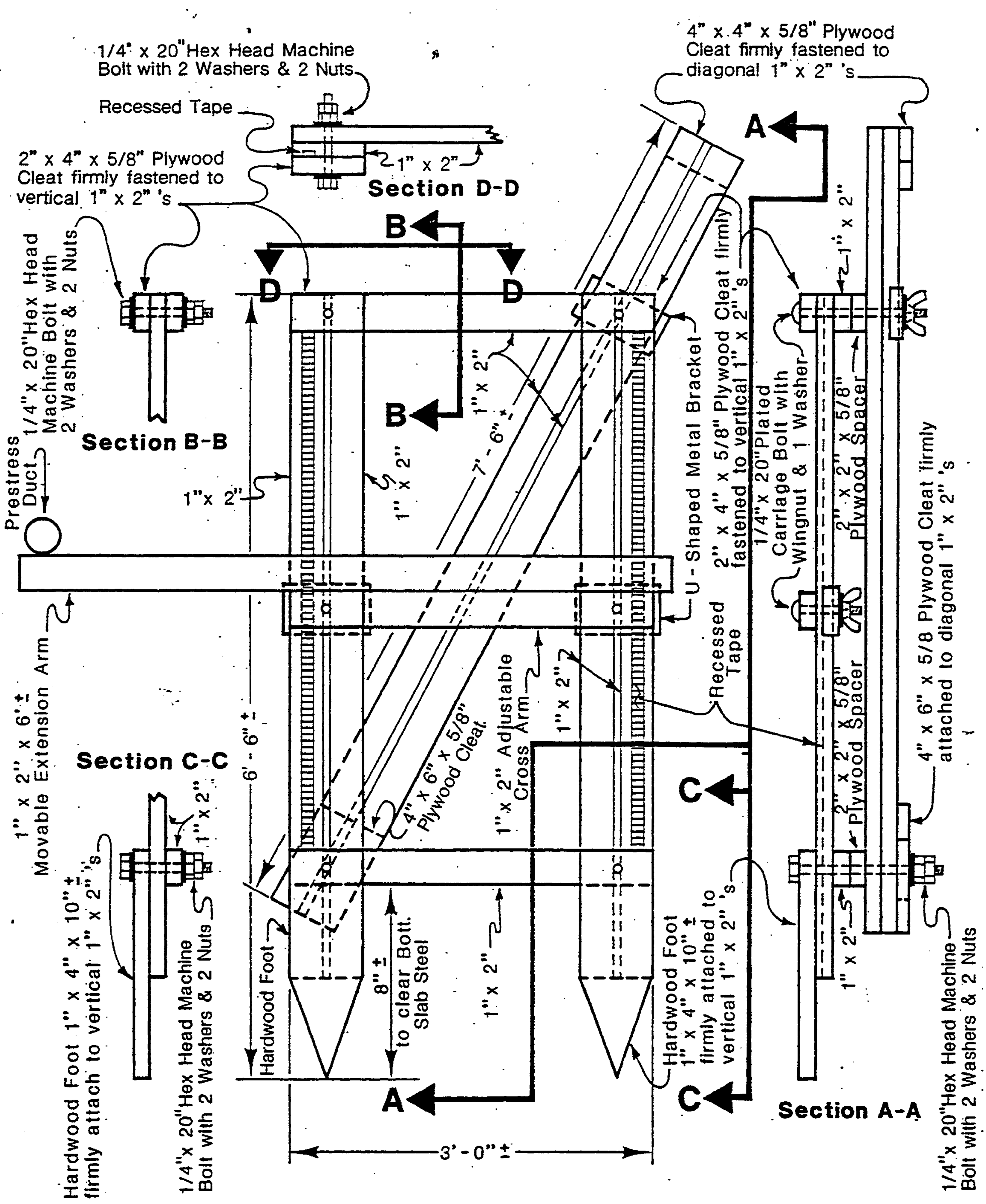
Attachment No. 1 to this Bridge Construction Memo is a sketch showing details for constructing a template for use in checking the profile grade of prestress ducts placed in post-tensioned, prestressed, concrete girders.

Attachment No. 2 to this Bridge Construction Memo describes the use of the template.

Although the template was designed primarily for use by the State's Engineer to check the duct profiles, it can also be used by the Contractor's ironworkers to set the ducts to the correct profile grade. It is understood that the use of the template can result in a considerable savings in time for contractor and state personnel, and reduce disagreements concerning the duct profile grade. The template is particularly useful for checking tendons in sloping exterior girders.

105F

ADJUSTABLE TEMPLATE FOR CHECKING PRESTRESS DUCT PROFILES



BRIDGE CONSTRUCTION MEMO 145-7.0
ATTACHMENT NO. 1 (11-19-84)

Sheet 1 of 1

106F

INSTRUCTIONS FOR USING THE ADJUSTABLE TEMPLATE FOR CHECKING PROFILES OF DUCTS IN POST-TENSIONED, PRESTRESSED, CONCRETE GIRDERS.

1. Set the template on the soffit forms adjacent to the girder where the height of duct is to be established or checked.

2. Plumb the legs of the template and tighten the wingnut at the upper corner of the template.

3. Place the movable extension arm on top of the adjustable cross-arm and adjust the height of the cross-arm. The top of the movable extension arm should be set at the same height above the soffit as is required for the bottom of the duct.

4. When the top of the movable extension arm is at the correct height above the soffit, tighten the two wingnuts in the adjustable cross-arm. (Note that the adjustable cross-arm will be on the same slope as the bridge soffit.)

5. Slide the movable extension arm along the top of the adjustable cross-arm until it contacts the girder side form. The bottom of the duct will be at the proper height when it is set to contact the top of the movable extension arm.

Volume II

MECHANICALLY STABILIZED EMBANKMENT WALL CONSTRUCTION CHECKLIST

The Mechanically Stabilized Embankment (MSE wall) construction checklist has been developed to serve as a resource to assist field personnel in quality assurance inspection and also to raise the awareness of maintenance inspection frequency.

There are several proprietary MSE systems; however, they all consist of four basic elements: select granular backfill, coated metal reinforcement (reinforcing strips or welded wire mesh), precast concrete or elliptical steel facing panels and concrete cap (most likely with all systems). A MSE wall is a composite material formed by the association of a frictional soil and reinforced metal strips or welded wire mesh. The strips or the meshes resist stresses produced within the soil mass. The stresses are transferred to the strips by friction. Each element of a MSE wall can influence the overall performance of the structure. The soil reinforcement matrix is the key structural element of the system. The facing must be sufficiently rigid to resist the applied earth pressure and have the flexibility to tolerate differential settlement without structural distress. The select granular backfill must be well drained to relieve the built-up hydrostatic pressure behind the back of the facings, if not this may cause settlement.

The MSE Walls must be maintained and inspected frequently to assure the performance throughout the design life of the system. MSE Wall maintenance inspection program was established in 1986 to monitor the corrosion rate of the steel reinforcement elements used in MSE walls. Steel inspection elements are installed in all MSE walls for subsequent extraction and corrosion evaluation. Sufficient inspection elements are placed so samples can be removed from each wall after 5, 10, 20, 30, 40, and 50 years. The inspection program requires several units to coordinate effort to remove, test and archive test results. Structure Maintenance and Investigations is responsible for removal of the test steel reinforcement elements and archiving the result. Geotechnical Services provides assistance with inspection element extraction, while Materials Engineering and Testing Services (METS) conducts the corrosion evaluation of the inspection elements.

I. SOURCES OF INFORMATION

A. Bridge Construction Records and Procedures Manual – Chapter 145
B. Bridge Design Aids – Chapter 3-1.1
C. Caltrans Highway Design Manual – Chapter 200, Topic 210

II. SPECIFICATION REQUIREMENTS:

A. Contract Special Provisions
B. Contract Plans
C. Standard Specifications

1. Portland Cement, Section 90
2. Engineering Fabrics, Section 88
3. Earthwork, Section 19
4. Permeable Materials, Section 68-1.025
5. Concrete Structures, Section 51
6. Minor Concrete, Section 90-10
7. Galvanizing, Section 75-1.05
8. Reinforcement, Section 52
9. Mortar, Section 51-1.135
10. Elastomeric Bearing Pad, Section 51-1.12H
11. Corrugated Metal Pipe, Section 66
12. Underdrains, Section 68-1
13. Prestressing Steel, Section 50-1.05
14. Plans and Working Drawings, Section 5-1.02
15. Water, Section 90-2.03

III. ITEMS TO BE RECORDED IN THE JOB FILES

A. Reports of Inspection of Materials (Category 41)
   1. Reinforcing steel strips or welded wire mesh and fasteners
   2. Precast facing panels
   3. Elastomeric bearing pads
   4. Structure backfill materials
   5. Engineering filter fabrics
   6. Geodrain materials, if used

B. Contractor's Submittals (Category 12)
   1. Shop Drawings
   2. Excavation Plans, if needed
   3. Test data, if required
   4. If splices for soil reinforcement are used, the splice connection detail must be tested and submitted to Structure Design for approval

IV. PROPRIETARY EARTH RETAINING SYSTEMS:

The four pre-qualified proprietary mechanically stabilized earth (MSE) systems, which have been pre-approved for use on State projects, are:

System	**Suppliers**
Retained Earth	Foster Geotechnical 1660 Hotel Circle North - Suite 304 San Diego, CA 92108
MSE Plus	SSL 4740 Scotts Valley Drive, Suite E Scotts Valley, CA 95066
Reinforced Earth	The Reinforced Earth Company 20381 Lake Forest Drive, Suite B2 Lake Forest, CA 92630
Welded Wire Wall	Hilfiker Retaining Wall P.O. Box 2012 Eureka, CA 95502

However, the contract Special Provisions usually gives the contractor the option of choosing one of the proprietary systems as approved and listed in the Department's current list of pre-qualified earth retaining systems and is limited to only those systems determined to have characteristics suitable for each particular project. To obtain a current list of approved systems, contact Lily Sun, MSE wall technical specialist at 916-227-8615.

Systems that are not specified and not on the Department's current list must have the full range of parameters for which pre-qualification is required performed before approving the system for a particular project.

V. CONSTRUCTION

A. THE STRUCTURE REPRESENTATIVE IS RESPONSIBLE TO:

1. Send a 60 pound sample of backfill material to METS in Sacramento for each level where inspection elements are installed.
2. Send one 5-foot long representative sample of the soil reinforcement material to METS in Sacramento for testing.
3. If the soil reinforcement material is galvanized, measure and record the thickness of the zinc coating along the length of the least of 3 or the total number of permanent reinforcing elements at each level.
4. Obtain certified mill test reports and a Certificate of Compliance for the soil reinforcement steel and galvanizing.

B. BEFORE CONSTRUCTION:

1. Set up a pre-construction meeting to discuss any potential problems and/or the level of competency of the Contractor.
2. Remind the Contractor to submit wall type and working drawings to the Structure Design - Document Units in Sacramento. Only one type of wall will be used at all wall locations in a project.
3. Ensure the Contractor verifies the field dimensions and elevations prior to preparing the working drawings. This will mitigate any potential problems that may arise due to the precast panels being too short or too tall.
4. Remind the Contractor to submit the construction staking for the wall layout line, beginning of wall, end of wall and respective elevations.
5. Review shop drawings and verify the top of wall elevations versus the district grid grades, drainage plans and underground utility plans. Verify the locations of strips or wire mesh to ensure no conflict with any utility locations.
6. Confirm with the Contractor on material procurement, sampling and scheduling.
7. Coordinate with the District Landscape Architect, the Structure Designer, MSE Wall Specialist and the METS Material Engineer on review and approval of MSE wall type. When the submitted shop drawings are approved with minor corrections, the Structure Representative is responsible to assure the Contractor incorporates such details in the final construction.
8. Review and approve the final texture of the facing panel and concrete mix design.
9. Review and approve the erection procedures.
10. Review and approve excavation plan if required.

C. DURING CONSTRUCTION: ***<u>THIS IS FULL-TIME INSPECTION</u>***

1. Check sloping or shoring installation if required.
2. Ensure the leveling pad is constructed properly. Check against the reference stakes for location and elevations. An improperly placed leveling pad can result in subsequent panel misalignment, and decrease wall construction productivity.
3. Check materials to assure quality: cracks in panels, rust on strips, panels with uneven edges, etc. Since the metal strips are prone to corrosion, which reduces the capacity of the unit, galvanize or epoxy-coating thickness must be checked to make sure there was no damage during shipping and installation. All materials incorporated into the work must be inspected thoroughly prior to placement. Once constructed, it is difficult to remove and replace. Ensure the Contractor stores materials properly. Polyurethane foam and filter fabric must be stored away from sunlight.
4. Collect Certificates of Compliance and any samples as required. Take photos of release tags that are stenciled on the backside of the panel.
5. Ensure the Contractor provides adequate equipment to lift and set panels. Each 5 1/2” panel typically weights 1700 lbs. The heaviest can weight 3000 lbs.
6. Ensure the wall LOL is straight and at the right batter, as specified in the shop drawings, to account for the backfill displacement.
7. All vertical joints must be protected with engineering filter fabrics prior to backfilling
8. Ensure the Contractor installs drainage system per plan before backfilling.
9. Ensure backfilling is done with proper equipment to avoid damage to the panels. Sheepsfoot rollers are not to be used for compaction of select granular backfill within the limits of the soil reinforcement. Handheld or hand-guided compacting equipment shall be used within one foot of the facing panels.
10. Space bars must be placed between two adjacent panels to maintain the spacing of the interlocked panels.
11. Panels must be given a slight batter toward the backfill, as indicated on the shop drawing, to compensate for outward movement caused by placement and compaction.
12. Number of reinforcing strips per panel may vary throughout the length of the wall. Ensure the Contractor places the panels correctly. It is of utmost importance that the panel type and number of tie strips in the panel coincide with the requirements shown on the approved plans.
13. Reinforcing strips must be placed in the proper alignment and on the level surface, usually an inch higher to avoid the downdrag by settlement of the backfill after compaction. Ensure the locations of test strips are centered in the opening of the panels.
14. Check condition of the reinforcing steel strips prior to backfilling.
15. Reinforcing steel strips shall be installed perpendicular to the back of the facing panel, with the proper density and length of strips being attached to each panel.
16. Backfill material shall be sampled and tested for gradation and moisture content. It is critical to control the backfill quality to avoid excessive settlement that may eventually compromise the integrity of the system.

17. Fabrication of any top face panels under a traffic barrier shall be delayed until all lower levels of panels have been placed and any required settlement period has elapsed.
18. Once the system is complete, a concrete cap will be formed and placed. Ensure all reinforcing steel for the barrier rail is placed before concrete placement.
19. Check wall cap profile elevations to match the cross section and super-elevations of the roadway section.
20. Ensure the bridge number is painted on the face at the appropriate location. For on-going contracts without an assigned bridge number, contact DSMI for a bridge number and paint it on the structure at the appropriate location. The bridge number for any MSE wall shall have an “M” suffix.

D. POTENTIAL PROBLEMS:

1. The following cases of distortion may be seen in walls:
   a. Differential settlement that causes panels to contact each other resulting in chipping or spalling
   b. Low spot in wall
   c. Overall wall leaning

      These conditions may be due to weak or improper bearing foundation material. Ensure the quality of the foundation materials and proper compaction. In addition, the leveling pad should be constructed within the allowable tolerance per the shop plans.
2. When the wall is leaning out, the following conditions shall be checked:
   a. Panels not battered sufficiently
   b. Large backfill placing and/or compaction equipment working within 3’ zone of back of wall
   c. Backfill material dumped close to free end of reinforcing strips, then spread towards back of wall, causing bulge in strips and pushing panels out
   d. Backfill material too wet
   e. Backfill contains excessive fines materials (beyond the Specifications for percent of materials passing a No. 200 sieve)
   f. Backfill material pushed against back of wall before being compacted on strips
   g. Excessive or vibratory compaction on uniform fine sand (more than 60 % passing a No. 40 sieve)
   h. Wedges not seated securely
   i. Clamps not tight
   j. Excessive lift thickness
   k. Plasticity index of backfill material in excess of specification limits
3. When the wall is leaning in, the following conditions shall be looked at:
   a. Excessive batter set in panels for select granular backfill material being used
   b. Inadequate compaction of backfill
4. Localized differential distortion may occur when adjacent panels have different battered angles.
   a. Create points of inflection and excessively wide joints

b. Allow the filter fabric to be exposed to ultra-violet light through the open joints, resulting in its degradation, which leads to the loss of fines behind the wall.

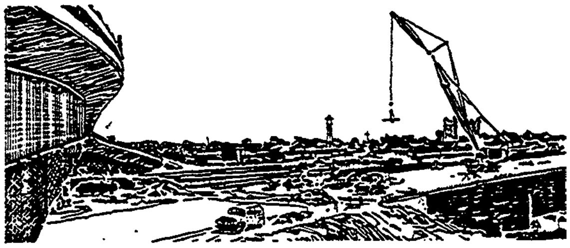
An identified cause is when a piece of heavy equipment turns while atop the wire meshes or reinforcing strips. This action tends to transfer a torsional pull to the reinforcing elements that may cause the panels out of alignment. The Contractor should submit a mitigation plan if such condition continues to occur. The above construction defect, although not necessarily structurally deficient nor aesthetically unacceptable it could create long term maintenance problems and potentially reduce the service life of the structure.

D. AFTER CONSTRUCTION/FINAL WALK THROUGH

1. Invite responsible Area Bridge Maintenance Engineer (ABME) to attend the final walk through.
2. Check that all test strips are in place according to the contract plans.
3. A Structure Maintenance & Investigation representative will initiate the ORIGINAL REPORT for maintenance.

V. FINAL RECORDS

1. After the MSE structure is relieved of maintenance, the Structure Representative must send of copy of Relief of Maintenance along with the Report of Completion and copies of all test reports including mill test reports and certificates of compliance to Structure Construction - Sacramento Office.
2. The Structure Representative shall submit the As-Built plans Structure Construction - Sacramento Office at the completion of the project or after relief of maintenance of the structure. All minor corrections as noted on the approved shop drawings, if any, shall be incorporated and finalized on the as-built plans.
3. The locations of test strips, soil resistivity parameters, galvanizing thickness must be logged on the As-Built plans.



BRIDGE CONSTRUCTION MEMO 145-9.0
MISCELLANEOUS INFORMATION
May 20, 1987

Volume II

REVIEW OF WORKING DRAWINGS FOR PROPRIETARY EARTH RETAINING SYSTEMS

When earth retaining systems are to be constructed the Contractor is allowed the option of a State designed system which is completely detailed on the plans or a pre-approved proprietary system which will be listed in the Special Provision.

Memo to Designers 5-16, which is Attachment No. 1 to this memo, covers procedures for plan submittal and checking of working drawings for the proprietary systems. In that memo a reference to Memo to Designers 11-l is made. It can be found in Volume II of the Bridge Construction Records and Procedures in Memo 160-6.0.

The Special Provisions require submittal of 4 sets of plans from the Contractor to Structure Design. Two unchecked sets will be sent to the Structure Representative. One set shall be given to the Resident Engineer. It is important that any field changes in alignment, grade, earthwork, traffic handling, etc. which may affect the wall design be reported to design as soon as possible prior to final approval.

After plans are approved 3 sets will be sent to the Structure Representative. He will give one set to the Contractor's Field Representative.

Starting about July 1987 special -provisions will require the Contractor to submit proprietary wall plan tracings to Structure Design upon receiving notice of plan approval. As Built plans will be printed and sent to the Structure Representative who will record changes and send to design at the completion of the job along with other As Builts.

During construction, the proprietary wall shall be built using the approved plans and sheets or details of the State designed system which are referred to by the approved plans. In general the State designed plans are superseded by the proprietary wall plans except for those sheets or details which are used by reference.

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REVIEW OF WORKING DRAWINGS FOR PROPRIETARY EARTH RETAINING SYSTEMS

Current bridge design practice allows the contractor the option of selecting a State designed earth retaining structure which is completely detailed on the contract plans or a pre-approved proprietary earth retaining system which is listed in the special provisions.

Acceptance of a proprietary system requires the submittal of complete working drawings for review and approval, and microfilm of the working drawings after completion of the project.

- Procedure

Applicable instructions under the Procedure section of Memo to Designers 11-1, Review of Working Drawings Prestressed Concrete, will apply to the review of working drawings for proprietary systems with the following modifications:

The distribution of the initial four sets of working drawings by the Documents Unit is as follows:

> Two sets to the Structures Representative, one set of which the Structure Representative will send to the District Resident Engineer or District Project Engineer. One set to the Substructure Committee technical specialists for review, comment and forwarding to the Design Section. One set to the Design Section.

The distribution of any subsequent submittal prior to approval is as follows:

> One set to the Structure Representative and remaining sets to the Design Section.

For each submittal of working drawings, including the submittal finally approved, the Design Section should retain one set. The Documents Unit will retain in the job file one set of approved working drawings.

- Guide for checking working drawings

The working drawings for the proprietary system should be checked from the perspective that the drawings have sufficient detail to construct the system at the correct location; have adequate dimensions; use the appropriate materials; take into account all items of contract work that may be affected by its construction; and conform to the pre-approved design and details.

The following items are pertinent to checking a proprietary system:

1. Review the Contract Plans and Specifications thoroughly.

2. The design and details information for the proprietary system can be obtained from the Substructure and Retaining Walls Committee. This information should be returned as soon as the review of each submittal is completed.

3. The Structure Representative and the District Project Engineer should be contacted early in the review of the initial submittal to determine if any changes have been made, or are pending, in the contract work that will affect the requirements of *the* proprietary system. The Structure Representative should be reminded that Design will check the working drawings for structural adequacy and construction sequences that affect the design. Items that affect traffic, excavation and safety should be evaluated by Construction, keeping in mind that temporary shoring may be required to construct the proprietary system and satisfy all contract requirements.

4. The special provisions require the contractor to verify existing ground elevations. If existing conditions indicate that the horizontal and vertical limits of the State-designed wall need to be revised, if built, a Contract Change Order will be required.

5. Contract items that affect the proprietary system, such as railings, sign structures, electroliers, utilities, approach slabs and connections to other structures, require special details to be included on the working drawings. Details of these items have not been pre-approved and require the submission of design calculations with the working drawings.

6. Most of the proprietary systems incorporate materials that are not covered in the Special Provisions, such as systems using soil reinforcement. The working drawings should indicate the ASTM designation and/or the particular grade of all material not otherwise covered in the specifications. This information is necessary for Translab to inspect and release the proper material and for the Structure Representative to accept at the job site.

7. The Special Provisions may require physical testing of portions of the proprietary system as a condition of acceptance. When such a test is specified, obtain the test results from the Structure Representative and evaluate whether or not they ate acceptable.

8. The Special Provisions may include requirements that modify the pre-approved design and details. A common example of this is the base width requirement which may be revised for external stability considerations. The Substructure and Retaining Walls Committee should be consulted if there is any question *of* interpretation of the pre-approved design and detail information.

9. Proprietary systems using soil reinforcement require the installation of inspection wires at the same location as the State designed MSE wall. Inspection wire locations should be shown on the working drawings.

Guy D. Mancarti
Design Supervisor

Philip C Warriner
Design Supervisor

TPJ/JCM

BRIDGE CONSTRUCTION MEMO 145-9.0
ATTACHMENT NO. 1 (5-20-87)
Sheet 2 of 2

Volume II

TIEBACK WALL CONSTRUCTION CHECKLIST

In conjunction with Chapter 9 of the Trenching and Shoring Manual and Chapter 11 of the Foundation Manual, a construction checklist for tieback wall has been developed to assist field personnel in preparing documents and inspecting field work to ensure compliance with contract requirements. It is important that the Structure Representatives review the plans and specifications and conduct preconstruction meetings with the contractor to lay out the procedures, identify field problems, and other issues.

Tieback walls are usually constructed in treacherous terrain with steep slopes and unstable footings; thus working safely is critical. The Contractor shall comply with the CalOSHA Construction Safety Orders and Storm Water Pollution Prevention Plan to reduce potential impacts to the site.

Structure Representatives are encouraged to employ the following checklist for tieback wall construction. Contact the Wall Specialist in the Division of Structure Design in Sacramento for additional assistance.

I. SOURCES OF INFORMATION

A. Bridge Construction Records and Procedures Manual – Chapters 145, 160
B. Foundation Manual – Chapter 11
C. Trenching and Shoring Manual – Chapter 9 –General information on tieback theory

II. SPECIFICATION REQUIRMENTS

A. Contract Special Provisions
B. Contract plans
C. Standard Specifications
   1. Testing by Contractor, Section 6-3.02
   2. Structure Excavation and Backfill, Section 19-3
   3. Piling (for the steel piles and drilled holes where they are placed), Section 49
   4. Prestressing Concrete (all sections except 50-1.07 apply), Section 50
   5. Shotcrete, Section 53
   6. Timber Structures (for the lagging), Section 57
   7. Preservative Treatment (for the lagging), Section 58
   8. Painting, Section 59

III. ITEMS TO BE RECORDED IN THE JOB FILES

A. Reports of Inspection of Materials (Category 41)
1. High Strength bars/strands
2. Sheathing (corrugated sheathing thickness, smooth sheathing material)
3. Corrosion Inhibitors
4. Anchor heads, wedges
5. Bearing plates and galvanized welded pipe extension

B. Contractor's Submittals (Category 12)
1. Shop Drawings
2. Hydraulic jack calibration chart and date of calibration
3. Theoretical elongation calculations
4. Welding Quality Control Plan
5. Pile Placing Plan

III. WHAT SHOULD BE DISCUSSED PRIOR TO THE FIELD WORK

1. Review Log of Test Boring and other geotechnical information for ground condition.
2. Discuss geotechnical design issues with the geologist and the project designer.
3. Remind the Contractor of his responsibility to submit shop plans, calculation sheets, and notice of material sources in a timely manner.
4. Ensure the designer approves the shop plans before any fieldwork begins.
5. Review approved shop drawings for bond length, hole diameter, grouting sequence, anchorage assembly, splice detail (when using HS bars), stressing sequence and proof and performance tests.
6. Coordinate with METS and verify source inspection and release of materials.
7. Coordinate with the Contractor to assure casing is provided if caving is anticipated.
8. Discuss with the Contractor and Caltrans Geologist the possible need for use of a "grout sock" in case of drilling through unfavorable conditions such as shattered rock, fractured formation or high ground water. A "grout sock" is a porous layer of filter fabric or equivalent that is lined the wall of the hole. It is inserted into the hole or inside the temporary casing prior to the grout is pumped into the hole. This is to prevent the excessive loss of grout into cracks, fissures or drainage gallery encountered.
9. Inspect site condition for equipment and crane setup. If a suspension platform is used, request and review information for load capacity requirements.
10. Discuss the locations of survey stakes and reference points to be provided by the Engineer with the Contractor.
11. It is critical to drill tieback holes at the proper angle or the contractor could have trouble during the stressing operation. Discuss sequence and operation of the following with the contractor tieback drilling, placing tiebacks, initial grouting, testing, final grouting procedures.

12. THE TESTING PROCEDURE IS VERY IMPORTANT. Discuss in detail how the Contractor will perform the Proof and Performance tests, i.e. the temporary stressing chair, use of shims, recording the elongation using a micrometer, etc. Emphasize that the testing procedures as specified (for time and load) will be strictly enforced to accurately measure the creep in the system. No deviations are allowed for such testing.
13. Ensure the concrete mix design, submitted by the contractor, is approved for use.

V. WHEN MATERIALS ARRIVE AT THE JOB SITE

1. Collect release tags on the tieback and anchorage assemblies and verify lot number with Form MR-29.
2. Inspect for cracks on sheathings and check for sheathing nominal thickness as specified.
3. Check condition of tendons and make sure rust inhibitor, if required, has been applied (randomly pick one or two anchors per lot and cut open the corrugated sheathing near the bonded/unbonded length to verify the PS strands are not rusted/pitted, and to also verify that the bonded length begins where it should, which is verifying the unbonded length also. Be careful not to knick the strand while cutting open an inspection window. One method would be to cut a 2”x2” or 3”x3” section of the sheathing on three sides and pull the section back. This makes it easy to repair by taping since the section is still intact with the sheathing).
4. Verify length of each tieback and make sure centralizers are installed adequately to avoid sagging.
5. If epoxy bars/strands are used, check coating for damage, cracking and other wears and tear.

V. DRILLING HOLE AND PLACING TIEBACK UNITS

1. Use survey stakes and reference points to verify wall LOL and tieback layout at every lift to assure conformance with the plans.
2. Verify drilling method, whether rotary or percussion. Keep in mind, vibration may cause unwanted settlement or raveling of the excavated face.
3. Verify drill bit diameter. Contractors tend to use drill bit of 1” smaller in diameter than the hole diameter. If drilling into sandstone or rock, the Contractor may need to use a drill bit size closer to the hole diameter because there will be less oversizing effect due to the percussion and blowing force of air.
4. Ensure the angle and diameter of drilled holes is in accordance with the plans (very important).
5. If ground water or soft soil is encountered, the Contractor is required to provide measures to protect holes from caving in.
6. If boulders or similar objects are encountered, the Contractor must provide measures such as down hole ram to pulverize and advance the hole to tip. Caution must be taken to avoid soil settlement, which may cause holes to collapse.

7. Ensure the hole is drilled to the specified depth and is clean of debris prior to installation of tieback unit.
8. The Contractor shall complete installing tieback units and grouting prior to drilling of the next row. This is to prevent collapsing of ungrouted holes. It is best if the grouting is performed the same day as the anchor is installed in the hole, if not immediately after placement. If the Contractor elects to install a certain number of anchors in the drilled holes and grout the following day, they must verify that the hole has not collapsed overnight. This is accomplished by pulling the anchor out of the drilled hole by a few feet and then having them push it back to the proper depth. If the hole has caved, the Contractor will either not be able to pull the anchor out or get it back to the proper depth. This is to be done on each anchor that is not grouted the same day as placement.
9. Check that centralizers are used to ensure proper center alignment of tieback unit.
10. Check grout consistency with flow cone (CA Test 541).
11. Grout tube must be inserted at the bottom of hole and gradually pulled up to avoid the development of air pockets and/or to displace any groundwater in the hole.
12. Verify that proper grout levels are achieved, may have to top off due to settlement of grout. Always make sure grout tube is extended a couple of feet into initial grout before topping off.
13. Check depth of grout for bonded length to assure conformance with the plans.
14. EQUIPMENT SETUP ARRANGEMENT:

    a. Ensure equipment is set up on firm ground. Inspect working bench for settlement or sign of cracks prior to start every day.
    b. Fall protection such as a handrail or harness shall be provided as required by Cal OSHA Construction Safety Orders.
    c. Ensure the capacity is not exceeded if a suspended drill platform is used.
    d. Spoil must be hauled away or stored as required by SWPPP.
    e. High-pressure hose from the air compressor must be protected.
    f. Personal safety equipment should be worn at all times.

VI. PLACING ANCHORAGES AND STRESSING

1. Prior to installation of bearing plate, check bearing surface conditions.
2. Blockouts must be sized to accommodate the jacking unit, per the shop drawing.
3. Check anchor head, wedges for rust, cleanness, cracks before installation.
4. Obtain Contractor’s jack information and verify with DMETS approved list for calibration date, gage factor and numerical display (accessible through DSC Intranet Web page).
5. Ensure performance test and proof test are performed in accordance with the required loads as specified in the contract Special Provisions. It is important that this is done properly, with the correct shim thickness.

6. Ensure the 10 minute constant loading be applied to measure the creep. If the creep exceeds 1mm in the first 10 minutes, the contractor is required to maintain the test load for an additional 50 minutes. The unit is rejected if it does not meet the two acceptance criteria (allowable creep and elongation) as stated in the job special provisions. Consult with the designer and geologist for an acceptable repair method.
7. Per the job special provisions, the Contractor is required to record the movement at the end of the tieback at each load increment during the load test. If the load test is held for 60 minutes, the movement of the end of the tieback shall be recorded at the specified time interval and a graph of creep vs. log (t) must be plotted for the movement between 1 minute and 60 minutes. During this time, the test load must be held constant. The plot of creep vs. log(t) is expected to be a straight line where the slope equals a constant c. (FOR MORE INFORMATION, REFER TO TRENCHING AND SHORING MANUAL, CHAPTER 9).
8. After the performance or proof test, the Contractor must set the unit back to the alignment load before proceeding to the final lock-off force as required by the Special Provisions. This procedure is usually performed as follows:
    a. With shim(s) in place, the Contractor brings the unit to the proof load (generally 150% T).
    b. Once the unit is at the proof load, the permanent wedges are then placed in the anchor head. The jack is released to transfer the load to the anchor head.
    c. Due to the anticipated loss in seating the wedges, the Contractor must restress the strands lightly and place additional shim(s) to accommodate this loss (usually 3/8”).
    d. The Contractor may have to perform a lift-off test to verify the permanent wedges are seated at the proof load (refer to BCM 145-11.0 for general information and concerns on anchor set and lock-off of epoxy coated tendons in the case where epoxy strands are used). This is done to assure the permanent wedges are properly seated and the ultimate capacity of the tieback unit is achievable.
    e. The contractor then removes the shim(s) to achieve the lock-off force. They then perform a lift-off test to verify the specified lock-off force is achieved. One method to check the lift-off force is to successfully remove a thin paper (without tearing it) that is wedged between the bearing surface and anchor head when the lock-off force is applied to the strands.
    f. This procedure may be reiterated for the first couple of units to adjust the shim thickness to achieve the required lock-off force. This is due to additional losses that may occur in seating the wedges and variations in the actual unbonded length.
    g. In any case, the Contractor is not allowed to stress the strands beyond the allowable 75% of the ultimate strength, unless otherwise stated in the contract Special Provisions.
9. If a head cap is required, ensure the Contractor installs and grouts the head cap adequately.

VII. WALL FACING CONSTRUCTION

1. Review contract plans for wall thickness, treatment details, profile elevations, and etc.
2. Review special provisions for prequalification and procedures if shotcrete facing is required.
3. Ensure drainage detail (geocomposite drain) is constructed per plan.
4. Ensure the shotcrete foundation is not disturbed, no sloughing or runoff.

IX. FINAL RECORDS

1. The As-Built plans shall be updated frequently during construction to assure adequacy and completeness.
2. At the end of the job, the Contractor is required to submit the as-built shop drawings to the Structure Representative. The SR reviews for accuracy and completeness.
3. The Structure Representative then submits the complete as-built drawing to DSD, Document Unit in Sacramento.

BRIDGE CONSTRUCTION BULLETIN **Approved:** FOR R. P. SOMMARIVA, Chief Office of Structure Construction	**File: 145-15** **MISCELLANEOUS INFORMATION** **Date: 6/12/98** Expires: None Supersedes: BCE Memo 83-5

Subject: Temporary Metal Anchorage Devices during Concrete Coring Operations

In connection with concrete coring operations, it has been noted that Contractors use metal anchorage devices to temporarily secure the coring equipment to the existing concrete structure.

The use of such metal anchorage devices is permissible provided they do not seriously damage the structure, and provided that the metal is removed to a depth of at least one inch below the surface of the concrete. Any resulting holes must be cleaned and filled with mortar.

It is not necessary to remove the anchorage devices or fill holes in cells of box girders or other similar interior locations.

c: BCR&P Manual Holders
Consultant Firms
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN

Approved:
R. P. SOMMARIVA, Chief
Office of Structure Construction

File: BCM 145-16
MISCELLANEOUS INFORMATION

Date: June 1, 1999
Expires: None
Supersedes: None

Subject: Selection of Random Samples

When sampling of material is required for job control tests, Office of Structure Construction (OSC) policy requires samples to be selected randomly (e.g. welds to be radiographed, ultimate splices to be sampled). To ensure randomness it is suggested that OSC personnel use a random number generator[1] to select the designated samples from a 'lot'.

Attachment 1 to this bulletin gives directions for using Microsoft Excel and the HP 32SII calculator in generating random numbers.

[1] A random number generator is available to all Structure Representatives within Microsoft Excel that is part of the Microsoft Office suite. It is recommended that the "RAND" function be used to obtain random numbers. Another random number generator is on the HP-32SII calculators.

c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager

Using RAND in Microsoft Excel to Generate Random Numbers

The following is an example of generating sample numbers for selecting four samples from a lot of 150 using the RAND function in Excel:

Excel offers users numerous methods to paste a function into the worksheet. For demonstration purposes, use one of the following methods for doing this.

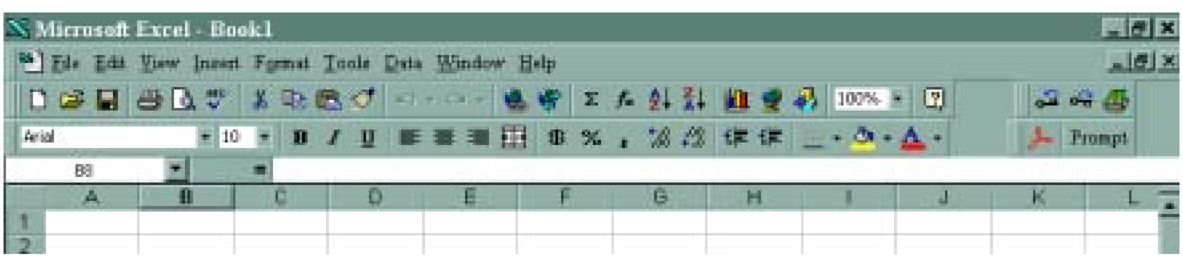
Method I

a. Select any cell within the worksheet.
b. Click on the "=" sign to the left of the formula bar,
c. To the left of this you will see the function box,
d. Clicking the arrow to the right of the function box will display a list of functions that were currently used, you are looking for the function labeled "RAND".
e. If you do not find "RAND" on the list, select the option "More Functions", this should give bring up the "Paste Function" dialog box.

Or

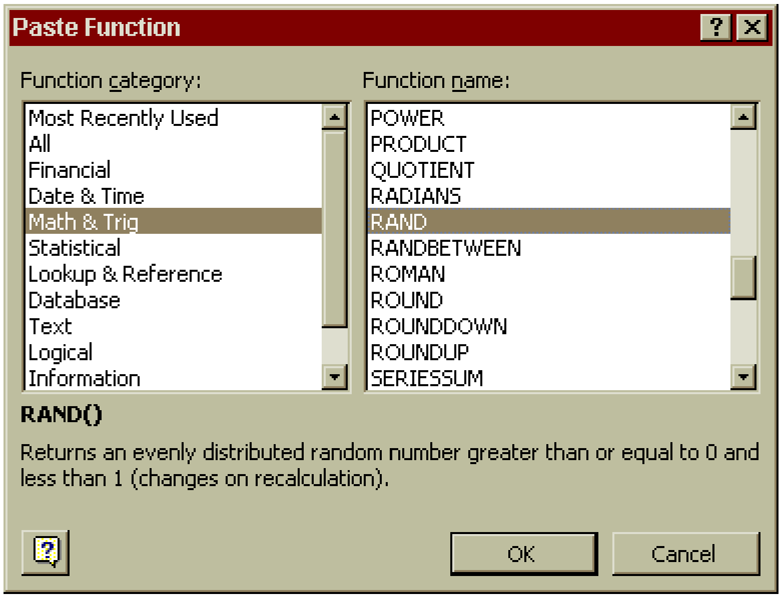
Method II

Click on the paste function button on the menu bar (*fx*), this should bring up the "Paste Function" dialog box.

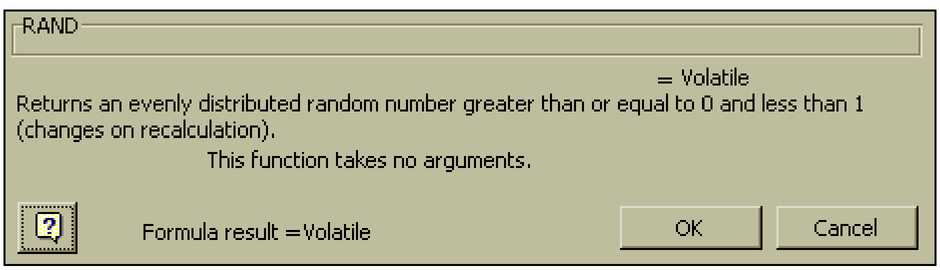


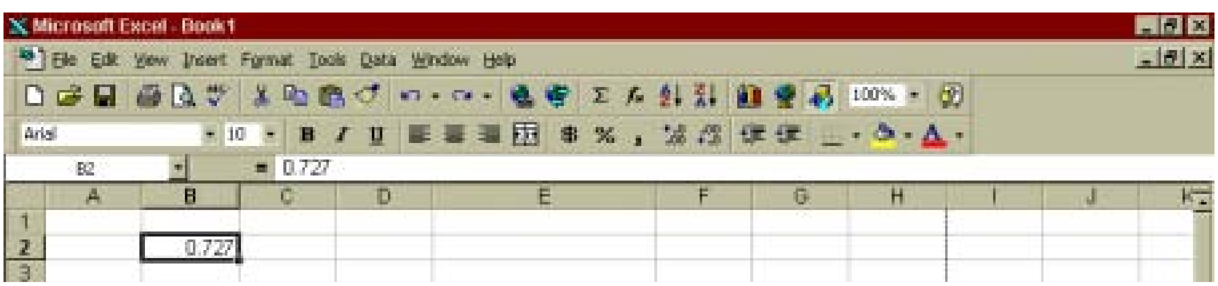
Part II

a. Click on "**Math & Trig**" in the "Function category" column.
b. Click "**RAND**" in the "Function Name" column.
c. click ok and that will place the function into the formula bar of your worksheet.
d. d. click ok again to accept this function. Set the format for your numbers as you normally would in Excel.



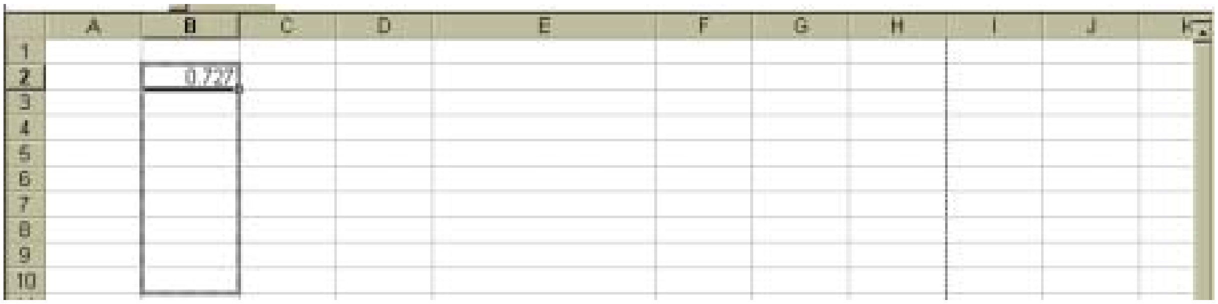
Clicking “OK” will place the RAND function into the selected cell and present this dialog box to you, click “OK” in this box and it will complete the function by placing a random number into your selected cell.



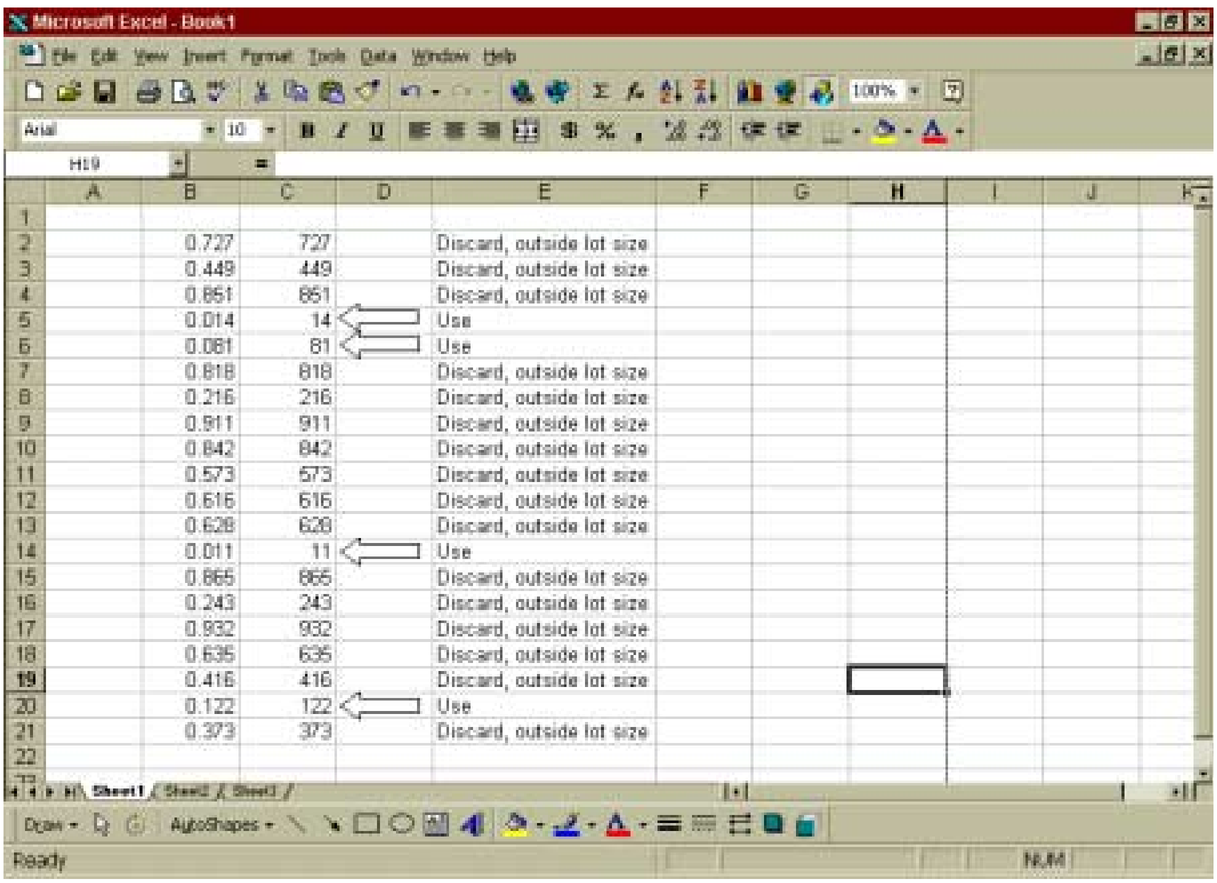


Create a list of random numbers by copying the cell with the “RAND” function to other cells. To do this you highlight the cell and then drag the corner down to fill whichever cells you need. When you release after dragging the “RAND” function will be copied into the desired cells.

Set the format to three decimal places and then equate these numbers as though there is no decimal point. Another method would be to multiply them by 1000 and read them directly.



The numbers used are below, the function is volatile meaning that it changes with each operation on the worksheet. Column B contains the “RAND Function” and Column C is the results multiplied by 1000.



	A	B	C	D	E
1					
2		0.727	727		Discard, outside lot size
3		0.449	449		Discard, outside lot size
4		0.861	861		Discard, outside lot size
5		0.014	14		Use
6		0.081	81		Use
7		0.818	818		Discard, outside lot size
8		0.216	216		Discard, outside lot size
9		0.911	911		Discard, outside lot size
10		0.842	842		Discard, outside lot size
11		0.573	573		Discard, outside lot size
12		0.616	616		Discard, outside lot size
13		0.628	628		Discard, outside lot size
14		0.011	11		Use
15		0.865	865		Discard, outside lot size
16		0.243	243		Discard, outside lot size
17		0.932	932		Discard, outside lot size
18		0.635	635		Discard, outside lot size
19		0.416	416		Discard, outside lot size
20		0.122	122		Use
21		0.373	373		Discard, outside lot size

For this example, a greater number of numbers were generated than the four needed in the event the number repeats or falls outside the lot size. In the case of a number that repeats or is outside the lot size, you must skip it and use the next number that is unique and fits the criteria, unique and within the lot.

Using the HP 32SII RPN Scientific Calculator to Generate Random Numbers

The HP 32SII calculator also has the ability to generate random numbers within the probability menu of the calculator. The RANDOM function uses a seed to generate a random number. Each random number generated becomes the seed for the next random number. Therefore, a sequence of random numbers can be repeated by starting with the same seed. If Structure Representatives use the HP 32SII calculator to generate random numbers, they are encouraged to change the seed number each time they start the process.

The following is an example of generating sample numbers for selecting four samples from a lot of 150 using the random number generator within the HP 32SII.

To generate random numbers with the HP 32SII, you use the function titled "PROB", it is located on the e^x key. Selecting "PROB" you will need to:

Display	Key stroke	
	➦ e^x	Start "PROB"
Cn,r Pn,r SD R	1/x	Set seed number, takes value from x register and uses it as the seed
0.000		Displays value of seed
	➦ e^x	Start "PROB"
Cn,r Pn,r SD R	Σ+	Generates random number
0.031		
	➦ e^x	Start "PROB"
Cn,r Pn,r SD R	Σ+	Generates random number
0.872		

Generate a list of random numbers using the RANDOM function in the HP 32SII. Set the format to three decimal places and then equate these numbers as though there is no decimal point. The start seed for this example is 0.

		Comment
0.031	**31**	**Use**
0.872	872	Discard, outside lot size
0.690	690	Discard, outside lot size
0.149	**149**	**Use**
0.167	167	Discard, outside lot size
0.134	**134**	**Use**
0.288	288	Discard, outside lot size
0.925	925	Discard, outside lot size
0.107	**107**	**Use**

For this example, a greater number of numbers were generated than the four needed in the event the number repeats or falls outside the lot size. In the case of a repeat or one that falls outside the lot size you must skip it and use the next one that is unique. The four samples that would be taken for this 150-lot size would be 31, 149, 134 and 107.

BRIDGE CONSTRUCTION BULLETIN **Approved: <u>Original Signed by Dolores Valls</u>** **DOLORES VALLS, Deputy Division Chief** **Offices of Structure Construction**	**File: BCM 145-17** **MISCELLANEOUS INFORMATION** **Date: July 30, 2002** **Expires: July 30, 2003** **Supersedes: None**

Subject: New Scaffolding Specification

The purpose of this Bulletin is to make all OSC personnel aware of the **new scaffolding specification**. All new construction contracts that require scaffolding placed over or adjacent[1] to public traffic will contain the new scaffolding Standard Special Provision (SSP). Typical examples of where this new SSP might be used are: work on existing structures, retrofit work, and bridge painting. The scaffolding ssp is not intended for work on sound walls, retaining walls or building contracts.

Attachment No. 1 is a copy of the new scaffolding specification. The specification requires working drawings to be submitted to the Engineer for all scaffolding that is constructed over or adjacent to traffic, or suspended from the traveled way. In addition, the scaffolding working drawings must be signed by a registered civil engineer and then independently reviewed, stamped, and signed by an engineer who is registered as a Civil Engineer in the State of California. The independent reviewer cannot be employed by the same entity preparing the working drawings. Upon receipt of a complete set of scaffolding working drawings, the RE/SR shall review and return the working drawings within one week. Written notification indicating that the working drawings are found acceptable shall be given to the contractor.

Structure Representatives on existing contracts that will require scaffolding over or adjacent to traffic, or suspended from the traveled way, need to evaluate if this new specification should be placed into the contract by a contract change order. Prior to writing a contract change order issue should be elevated to the Bridge Construction Engineer and discussed with the Area Construction Manager and the appropriate Headquarters Office Chief.

[1] A scaffolding system is considered “adjacent” if the height of the system is greater than the shortest distance from the scaffolding to the edge of the traveled way. (i.e., if the scaffolding fell the traveled way would be affected).

c: BCR&P Manual Holders
CMB - Consultant Firms
RPieplow, HQ Division of Construction

SCAFFOLDING

Scaffolding shall be defined in accordance with and shall conform to the Construction Safety Orders of the Division of Occupational Safety and Health and these special provisions.

If scaffolding is constructed for this project over or adjacent to traffic, or suspended from the traveled way, the Contractor shall submit to the Engineer working drawings for scaffolding systems in conformance with Section 5-1.02, "Plans and Working Drawings'" of the Standard Specifications, and these special provisions.

Scaffolding working drawings shall include the following items:

A. Descriptions, calculations, and values for all loads anticipated during the erection, use, and removal of scaffolding.
B. Methods and equipment for erecting, moving, and removing scaffolding.
C. Design details including bolt layouts, welding details, and any connections to existing structures.
D. Stress sheets including a summary of computed stresses in the (1) scaffolding, (2) connections between scaffolding and any existing structures and (3) existing load supporting members. The computed stresses shall include the effects of erection, movement, and removal of the scaffolding.

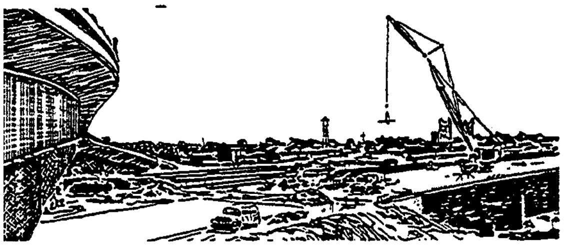
The scaffolding manufacturer's name, address, and phone number shall be shown on the working drawings.

The working drawings shall be stamped and signed by an engineer who is registered as a Civil Engineer. In addition, prior to submitting the working drawings to the Engineer, the working drawings shall be stamped and signed by an independent reviewer who is registered as a Civil Engineer in the State of California. The independent reviewer shall not be employed by the same entity preparing the working drawings.

The Contractor shall allow 1 week for the review of a complete submittal for scaffolding working drawings. In the event the Engineer fails to complete the review within the time allowed, and if, in the opinion of the Engineer, completion of the work is delayed or interfered with by reason of the Engineer's delay in completing the review, the Contractor will be compensated for any resulting loss, and an extension of time will be granted, in the same manner as provided for in Section 8-1.09, "Right of Way Delays," of the Standard Specifications.

Welding for the manufacturing and erection of scaffolding shall conform to the requirements in AWS D1.1 or D1.2 for steel or aluminum construction respectively.

Full compensation for conforming to the above requirements shall be considered as included in the contract prices paid for the various contract items of work, and no additional compensation will be allowed therefor.



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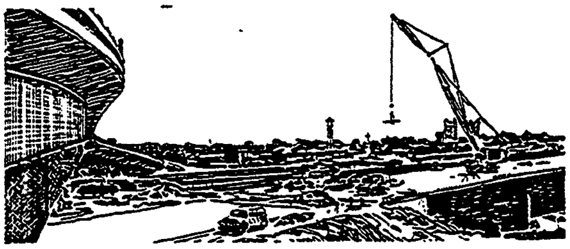
A. P. BEZZONE, Chief
Office of Structure Construction



OVERLOAD POLICY FOR CONSTRUCTION PROJECTS

Introduction

This Memo summarizes Office of Structure Construction policy with respect to administration of the "weight limitation" provisions of the contract. Adherence to the procedures as established herein will ensure uniform control of the movement of construction equipment over structures within the limits of highway construction contracts.

Standard Specification Weight Limitations

Section 7-1.02 of the Standard Specifications sets forth weight limitations for earthmovers, truck, and truck and trailer combinations which will be permitted to cross completed bridge structures, and in addition, provides that other construction equipment may be permitted to cross bridge structures subjects the weight limitations and conditions of the California Department of Transportation Permit Policy. Attachment #l of this Bridge Construction Memo presents the excerpts from the Department Permit Policy.

Attachment #3 of this Bridge Construction Memo presents charts "A" through "H" which may be used to determine whether or not any given two, three, four, five or six axle truck crane exceeds the allowable purple loading. These charts are based on the assumption that tandem or triden axles are spaced at 4'-6" centers for 3 and 4 axle cranes and 5'0" centers for 5 and 6 axle cranes. If the axles are spaced at some distances other than shown, the charts will be slightly in error. Attachment #3 presents instructions and sample problems which illustrate the procedure for using charts "A" through "H".

It is IMPORTANT that purple loads only be allowed on purple rated bridges. Some existing bridges on going contracts may be rated for green loading. Load ratings can be obtained from Structure Maintenance.

The provisions of Section 7-1.02 apply only within the limits of highway construction contracts. Operation of construction equipment on State highways beyond the limits of a construction project is governed by the California Vehicle Code, and movement of vehicles (including construction equipment) which exceed the size and weight limitations of the Vehicle Code is not authorized except under permit issued by the District permit section.

Overloads

overloads on bridge structures in construction contracts may be divided into two general categories: repetitive and occasional. The categories are separately discussed below.

A. Repetitive Overloads

Repetitive overloads usually occur in connection with an earthmoving operation, and usually involve earthmoving equipment.

1. Bridge Structures Designed for HS20-44, Alternative, and Permit Live Loading

In this case the limitations of Section 7-1.02 will apply and the use of earthmoving equipment which exceeds these limitations will not be permitted.

However, under the provisions of Section 7-1.02 the Contractor may request the redesign of a structure to increase its load carrying capacity.

At the present time, structures designed for overloads are based on a three-axle vehicle having a maximum axle load of 130 kips and a total gross load of 330 kips for spans greater than 54 feet, and a two-axle vehicle having a maximum axle load of 130 kips and a total gross load of 200 kips for spans of 24 to 54 feet. For spans under 24 feet, the design is based on a single 130 kip axle.

If the stresses produced by the Contractor's equipment do not exceed the stresses produced by the aforementioned overloads and if the Contractor is willing to pay the cost of redesign and the increased cost of construction, the structure or structures will be redesigned. (Additional information relative to construction overload design is given in Memo to Designers l5-15.)

Following are the procedures to be followed when the Contractor requests a redesign of a structure, or structures, to increase the load carrying capacity.

(a) Contractor submits a letter to the Resident Engineer requesting that the structure be designed to increase its load carrying capacity. In this letter the Contractor must name the structure or structures to be redesigned, give specific details of the loads and the positioning of the loads on the structure. He must also state that he is willing to pay the cost of redesign and the increased cost of construction.

(b) Structure Representative submits a copy of the Contractor's letter to the Chief, Office of Structure Construction along with a request that the structure be redesigned. The Structure Representative should also request that he be advised of the estimated cost of redesigning the structure.

(c) Upon receiving drawings for the redesigned structure and the estimated maximum cost of redesigning the structure, the Structure Representative will prepare a Contract Change Order authorizing the structural alterations to accommodate the construction overloads, and also stating the maximum cost to the Contractor for the redesign. (See Attachment #4 of this Construction Memo for a sample of this type of Contract Change Order.)

(d) When the work of constructing the redesigned structure is complete, the Structure Representative should write to the Chief, Office of Structure Construction, and request the actual cost of redesigning the structure.

(e) The actual cost of redesigning the structure, as obtained from the Office of Structure Construction, should then be deducted from the Progress Pay Estimate.

2. Bridge Structures Designed for Overloads

If a structure is designed to accommodate the 330 kip construction equipment loading, no further increase in load-carrying capacity will be considered and the use of construction equipment which exceeds the 330 kip loading will not be allowed.

If a structure is designed for more than the HS20-44 live load but less than the 330 kip construction equipment load, allowable overloads will be limited by the design load unless the Contractor requests a redesign. If a redesign is requested, the procedure will be the same as provided above for structures designed for HS20-44 live load.

B. Occasional Overloads

Occasional overloads will include the movement of construction equipment (cranes, paving equipment, etc.) across completed structures from one work site to another. Also included are stationary overloads, such as cranes operating from a bridge deck while placing concrete or performing other work.

1. Moving Overloads

If the structure is designed for HS20-44 and permit design live load and if the occasional overload does not exceed the "purple" loading, or in the case of structures designed for construction overloads, if the overload does not exceed the design overload, the overload may be authorized at the job level.

Overloads which exceed the "purple" loading, or which exceed the design loading if the structure is designed for construction overloads, must be referred to the Sacramento Office of Structure Construction.

2. Stationary Overloads

Requests to place stationary or static loads on completed or partially completed structures should be investigated to determine the capability of the structure to withstand the applied load.

If the Structure Representative is satisfied that stresses produced by a static load do not exceed the maximum allowable design stresses, approval may be given at the job level. If the effect of the load is not readily apparent or if there is doubt as to the stresses produced, the Sacramento Office of Structure Construction should be consulted.

Processing Requests to Move Overloads

As previously noted, requests from Contractors to utilize earthmoving equipment or to move overloads which do not exceed the limitations of Section 7-1.02 of the Standard Specifications may be approved at the job level. All other requests are to be forwarded by the Structure Representative to the Sacramento Office of Structure Construction for decision.

The Contractor's request must be explicit as to the nature of the overload and the conditions under which it will be moved.

Information required includes the type, make and model of equipment, the axle spacing, axle width out to out of tires, the axle load (obtained by scale weight if possible), the width and number of tires, operation conditions, etc.

Permission to cross bridge structures with earthmoving equipment which exceeds the maximum weight limitations set forth in Division 15 of the Vehicle Code, but which does not exceed the limitations set forth in Section 7-1.02 of the Standard Specifications will be granted by means of a letter to the Contractor from the Resident Engineer or Structure Representative. An example of such a letter is attached. (Attachment #5). Note that the conditions contained in the example letter are the minimum conditions under which permission will be granted. Other conditions may be imposed, depending on the particular job circumstances. To ensure uniform application of Division of Structures overload policy, letters to the Contractor should follow as closely as possible to the form of the example letter.

Permission to cross bridge structures with construction equipment (other than earthmoving equipment) which does not exceed the limitations of Section 7-1.02 will not require a letter unless there are special conditions under which permission is granted. However, if special conditions or limitations are to be imposed, they should be incorporated into a letter similar to the example letter authorizing *the* use of earthmoving equipment.

Since construction overloads will often affect areas of responsibility of both the District and the Office of Structure Construction it is important that both be fully informed. Particular care should be taken by the Structure Representative to insure that copies of all correspondence related to overloads are furnished to interested District personnel.

EXCERPTS FROM THE
DEPARTMENT OF TRANSPORTATION PERMIT POLICY

25.66 Overload Charts

The Office of Structures Maintenance has charted all bridges on the State Highway System and assigned capacity ratings to all structures which control the amount of overload that may be authorized under permit across the given structure. (See Plate 25-l - Chart for Computing Overload Permits). The various routes are classified as capable of carrying orange, green or purple loadings as detailed on District maps.

Allowable orange, green and purple loadings do not coincide with legal loading formulas. In addition, permit loads may involve bonus loading for extra axle width and extra tires, or "flotation" type tires. One "flotation" type tire of 18 x 19.5 min. size is consideredto be equivalent to two standard tires.

Application for loads in excess of any posted bridge load limits will be denied. Sections designated orange as limited by the structures, allow a maximum single axle load of 20,000 pounds or a maximum axle group load computed on the basis of formula 700 (L + 40), applied to any combination of axles in a distance of 18 feet or less except that the weight tables for 4' and 5' axle spacing (32,000 lb.) applies.

Sections designated as green, allow a maximum single axle load of 24,000 pounds or a maximum axle group load computed by the the formula 1.30 x 700 (L + 40).

Sections designated as purple, allow a maximum single axle load of 28,000 pounds or a maximum axle group load computed on the formula 1.50 x 700 (L + 40).

Exceptionally heavy loads such as being transported on nine or more axle combination units shall be approved through the Office of Structures Maintenance prior to issuance of a permit. Due to the time required in analyzing structural capacity of structures en route, a minimum of three days should be allotted for this approval.

Bridge loading charts shall be kept up to date and other Districts advised of changes.

The current Bridge List shows the overload ratings of each bridge. This can be used in conjunction with the charts in issuing permits. This overload rating is listed in the "Capacity" columns and the method of coding is described in the information section in the front of the Bridge List.

Axles for permit purposes shall be considered to be 8 feet wide. An axle less than 7 feet wide shall be allowed that percentage it is of 7 feet. For example, an axle 3.5 feet wide would be allowed 50 percent of the standard chart.

Tandems with axle spacing less than 3.5 feet shall be considered as a single axle for permit purposes.

25.67 Overchart Loads and Special Trucking Units

Office of Structures Maintenance approval must be obtained before issuing permits to transport loads exceeding the charted capacity under permit in unusual circumstances.

All special permits subsequently authorized for overchart loads, must designate the authorized axle loads, or group axle loads, and any special conditions prescribed.

In submitting necessary data for checking, a complete description of the vehicle giving all distances between axles, weight on each axle, and overlength and overwidth or underwidth dimensions is required by the Office of Structures Maintenance so that actual stresses can be computed.

In order to eliminate the possibility of routing overloads across a bridge soon to be posted, the structure should be considered posted for recommended load limits upon receipt of the bridge report recommending such posting. Permits will not be issued to exceed the posted limits. If no detour is available and temporary strengthening at applicant's expense appears feasible full details should be referred to the Office of Structures Maintenance as for overchart loads.

Office of Structures Maintenance will also rate all special heavy-hauling vehicles designed for overload operations under permit. Districts may furnish tentative capacity ratings if requested for design purposes but if time will allow the design data should be re-referred to Headquarters. When a special trailer is constructed, allowable loading ratings will be assigned by the Office of Structures Maintenance upon receipt of the details.

A booster type axle is one that may be adjusted to assume a portion of the load otherwise carried by adjacent axles. In the event of failure, its load is imposed on the adjacent axles. The use of air or hydraulically operated booster axle systems is not permitted under the permit policy (They are used only to meet legal loading).

25.68 II (B) Truck Cranes

Operations of overweight truck cranes under permit are basically controlled by the Standard overload chart (Plate 25-l). An exception to this policy is made for truck cranes with axle. width out to out of tires of 120" or more, and having 4 or more 14" minimum size tires per axle on rear tandem. These truck cranes may be allowed a bonus load or a load transfer as indicated in the following sections (1) thru (4):

(1) Three axle truck cranes qualified by tire size and axle width may be allowed up to 7,000 pounds bonus load for axles 2 and 3 providing the spacing between 1 and 3 is greater than 18'. When the Spacing between axles 1 and 3 is less than 18' the weight on axles l-2-3 is computed from the formula 1.50 x 700 (L + 40) + 7,000 for purple and 1.30 x 700 (L + 40) + 6,000 for green. The rear tandem shall not exceed 54,300 for purple and 47,000 for green.

(2) Four axle truck cranes may be allowed a load transfer from front to rear of up to 7,000 pounds providing the spacing between axles 2 and 4 is greater than 18 feet. When spacing between axles 1 and 3 or axles 2 and 4 is less than 18 feet, the weight on axles l-2-3 or axles 2-3-4 is computed from the formula 1.50 x 700 (L + 40) + 7,000 for purple and 1.30 x 700 (1 + 40) + 6,000 for green. The rear tandem shall not exceed 54,300 for purple and 47,000 for green.

Editor's Notes:

(When the spacing between axle 2 and 4 <u>OR</u> 1 and 3 is less than 18' *the* allowable load on 1+2+3 and 2+3+4 will be 1.5 x 700 (L + 40) + 7,000. A 7,000 pound load transfer may be made from front to rear before checking the dual or three axle combinations.)

(3) Five or more axle truck cranes may be allowed a load transfer from front to rear of up to 7,000 pounds providing the weight on any group of axles within 18 feet does not exceed 1.50 x 700 (L + 40) + 7,000 for purple and 1.30 x 700 (L + 40) + 6,000 for green. The rear triden or a 5 or 6 axle crane shall act exceed 59,500 for purple and 51,570 for green.

(4) Two axle truck cranes must conform to the present truck policy. No bonus or transfer of loads will be permitted. It is emphasized that all truck cranes not qualified for bonus or transfer loads must also conform to the present truck policy, using overload chart, Plate 25-l.

25.68 II (I) toads an Boom Dollies and Boom Trailers

The gross weight imposed on the highway by the wheels of any one axle of a boom dolly or boom trailer shall not exceed 18,000 pounds and the gross weight upon any one wheel or wheels supporting *one* end of such axle, and resting upon the roadway, shall not exceed 9,500 pounds.

The total gross weight with load imposed on the highway by any group of two or more consecutive axles of a boom dolly or boom trailer shall not exceed that given for the respective distance in the following table:

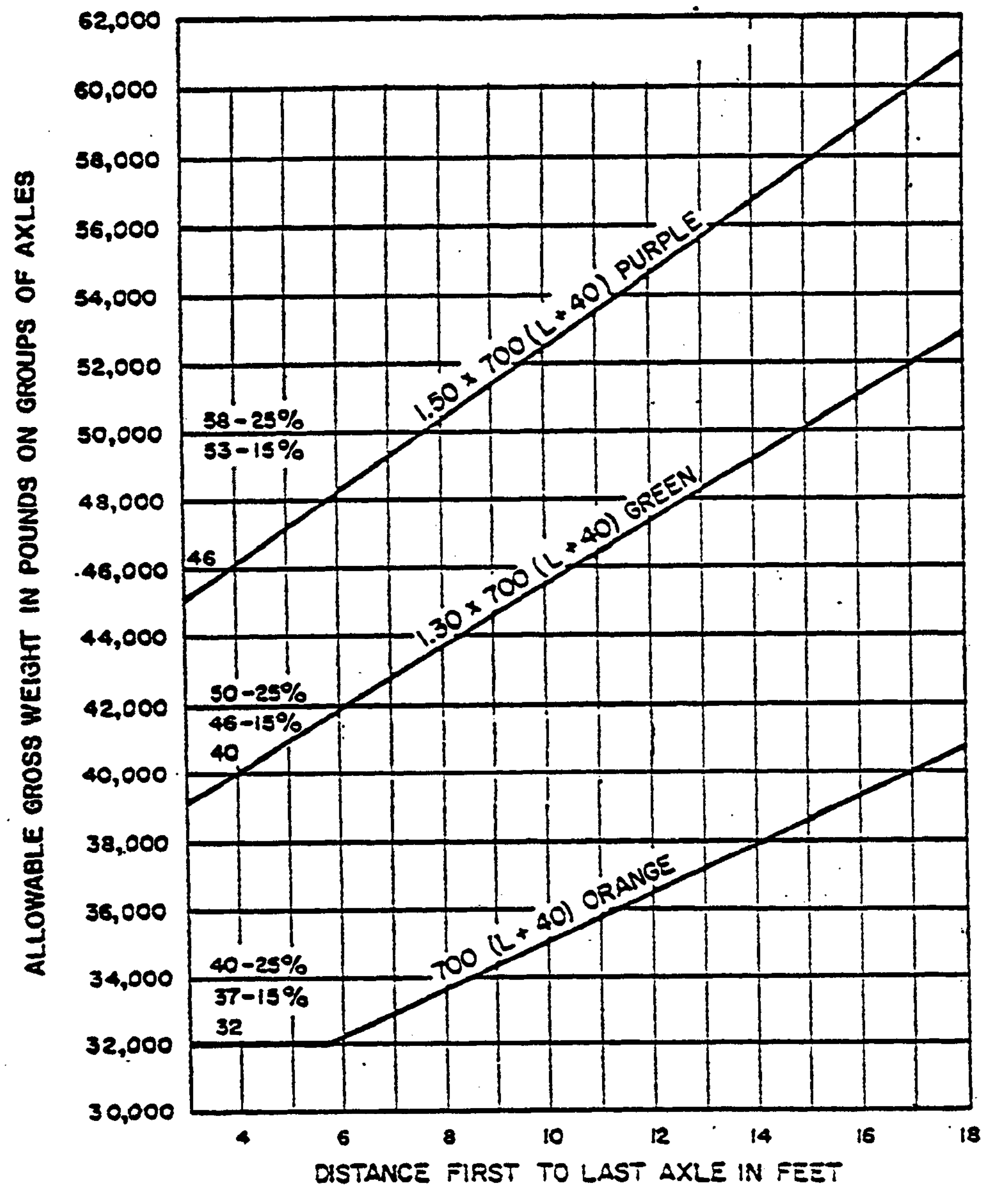
Distance in Feet Between First & Last Axle of Group	Allowed Load in Founds on Group of Axles
4	32,000
5	32,000
6	32,200
7	32,900
8	33,600
9	34,300
10	35,000
11	35,700
12	36,400
13	37,100
14	43,200
15	44,000
16	44,800
17	45,600
18	46,400
19	47,200
20	48,000
21	48,800
22	49,600
23	50,400
24	51,200
25	55,250
26	56,100

Boom support cables or hydraulic pressure must be fully released with weight of boom resting on dolly or trailer.

NOTE: The distance first to last axle of any 3 axle group combining crane carrier and the dolly must exceed 18 feet.

PLATE 25-1

CHART FOR COMPUTING OVERLOAD PERMITS



For vehicles with 8 tires per axle:

1. If gage is 8' ------ Add 15%
2. If gage is 10' or more -- Add 25%

Single Axle	
Purple	28,000
Green	24,000
Orange	20,000

PLATE 25-5
PURPLE AND BONUS OVERLOADS*

Example: 8'-0" First to Last Axle in Feet

50,400	4 tires, 8'-0" Wide	Purple Load = 1.5 x 700 (L + 40)
57,960	8 tires, 8'-0" Wide	Purple Load (+15%) = 1.15 x 1.5 x 700 (L + 40)
63,000	8 tires, 10'-0" Wide	Purple Load (+25%) = 1.25 x 1.5 x 700 (L + 40)

FT. \ IN.	0	1	2	3	4	5	6	7	8	9	10	11
2	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000
3	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	28,000 32,200 35,000	45,675 52,526 57,094	45,762 52,626 57,203	45,850 52,728 57,313	45,937 52,828 57,422	46,025 52,928 57,531	46,113 53,030 57,641
4	46,200 53,130 57,750	46,287 53,230 57,859	46,375 53,332 57,969	46,462 53,432 58,078	46,550 53,532 58,187	46,638 53,634 58,297	46,725 53,734 58,406	46,812 53,834 58,515	46,900 53,935 58,625	46,987 54,036 58,734	47,075 54,136 58,843	47,163 54,237 58,954
5	47,250 54,338 59,062	47,337 54,438 59,171	47,425 54,539 59,282	47,512 54,639 59,391	47,600 54,740 59,500	47,688 54,841 59,610	47,775 54,942 59,719	47,862 55,041 59,828	47,950 55,143 59,938	48,037 55,243 60,047	48,125 55,343 60,156	48,213 55,445 60,266
6	48,300 55,545 60,375	48,387 55,645 60,484	48,475 55,747 60,594	48,562 55,847 60,703	48,650 55,947 60,812	48,738 56,049 60,922	48,825 56,149 61,031	48,912 56,249 61,140	49,000 56,350 61,250	49,087 56,451 61,359	49,175 56,551 61,468	49,263 56,652 61,579
7	49,350 56,752 61,687	49,437 56,853 61,796	49,525 56,954 61,907	49,612 57,054 62,016	49,700 57,155 62,125	49,788 57,256 62,235	49,875 57,356 62,344	49,962 57,456 62,453	50,050 57,558 62,563	50,137 57,658 62,672	50,225 57,758 62,781	50,313 57,860 62,891
8	50,400 57,960 63,000	50,487 58,060 63,109	50,575 58,162 63,219	50,662 58,262 63,328	50,750 58,362 63,437	50,838 58,464 63,547	50,925 58,564 63,656	51,012 58,664 63,765	51,100 58,765 63,875	51,187 58,866 63,984	51,275 58,966 64,093	51,363 59,067 64,204
9	51,450 59,168 64,312	51,537 59,268 64,421	51,625 59,369 64,532	51,712 59,469 64,641	51,800 59,570 64,750	51,888 59,671 64,860	51,975 59,771 64,969	52,062 59,871 65,078	52,150 59,973 65,188	52,238 60,073 65,297	52,325 60,173 65,406	52,413 60,275 65,516
10	52,500 60,375 65,625	52,587 60,475 65,734	52,675 60,577 65,844	52,762 60,677 65,953	52,850 60,777 66,062	52,938 60,879 66,172	53,025 60,979 66,281	53,112 61,079 66,390	53,200 61,180 66,500	53,288 61,281 66,609	53,375 61,381 66,718	53,453 61,482 66,829
11	53,550 61,583 66,938	53,637 61,683 67,046	53,725 61,784 67,157	53,812 61,884 67,266	53,900 61,985 67,375	53,988 62,086 67,485	54,075 62,186 67,594	54,162 62,286 67,703	54,250 62,388 67,813	54,338 62,488 67,922	54,425 62,588 68,031	54,513 62,690 68,141
12	54,600 62,790 68,250	54,687 62,890 68,359	54,775 62,992 68,469	54,862 63,092 68,578	54,950 63,192 68,687	55,038 63,294 68,797	55,125 63,394 68,906	55,212 63,494 69,015	55,300 63,595 69,125	55,388 63,696 69,234	55,475 63,796 69,343	55,563 63,897 69,454
13	55,650 63,998 69,562	55,737 64,098 69,671	55,825 64,199 69,782	55,912 64,299 69,891	56,000 64,400 70,000	56,088 64,501 70,110	56,175 64,601 70,219	56,262 64,701 70,328	56,350 64,803 70,438	56,438 64,903 70,547	56,525 65,003 70,656	56,613 65,105 70,766
14	56,700 65,205 70,875	56,787 65,305 70,984	56,875 65,407 71,094	56,962 65,507 71,203	57,050 65,607 71,312	57,138 65,709 71,422	57,225 65,809 71,531	57,312 65,909 71,640	57,400 66,010 71,750	57,488 66,111 71,859	57,575 66,211 71,968	57,663 66,312 72,079
15	57,750 66,412 72,188	57,837 66,513 72,296	57,925 66,614 72,407	58,012 66,714 72,516	58,100 66,815 72,625	58,188 66,916 72,735	58,275 67,016 72,844	58,362 67,116 72,953	58,450 67,218 73,063	58,538 67,318 73,172	58,625 67,418 73,281	58,713 67,520 73,391
16	58,800 67,620 73,500	58,887 67,720 73,609	58,975 67,822 73,719	59,062 67,922 73,828	59,150 68,022 73,937	59,238 68,124 74,047	59,325 68,224 74,156	59,412 68,324 74,265	59,500 68,425 74,375	59,588 68,526 74,484	59,675 68,626 74,593	59,763 68,727 74,704
17	59,850 68,828 74,812	59,937 68,928 74,921	60,025 69,029 75,032	60,112 69,129 75,141	60,200 69,230 75,250	60,288 69,331 75,360	60,375 69,431 75,469	60,462 69,531 75,578	60,550 69,633 75,688	60,638 69,733 75,797	60,725 69,833 75,906	60,813 69,935 76,016
18	60,900 70,035 76,125											

* A set of tandem axles with spacing between axles of less than 3.5' is considered as a single axle (28,000 lb max)

PURPLE LOADS

GENERAL

Purple loading is a loading class established by the California Department of Transportation, Division of Maintenance and the office of Permits under the authority of Division 15 of the Vehicle Code. The loading is the maximum load class which is permitted under the Permit Policy by vehicles on Stats Highway Systems.

Purple loading is based on the following forumla:

Purple Load (lbs) = 1.5 x 700 (L + 40)

Where L = Distance in feet between the first and last axle Of any group of axles within 18 feet.

Plate 25-1 of the Permit Policy shows a line graph of this loading.

Plate 25-5 of the Permit Policy is a table showing values for all group axle spacings, each inch up to and including 18 feet. This is the best chart to use to check any vehicle (except cranes 120" wide out to out of tires and with 4 or more 14" minimum tires on each axle of the rear tandem a triden).

Plate 25-5 can be used as follows:

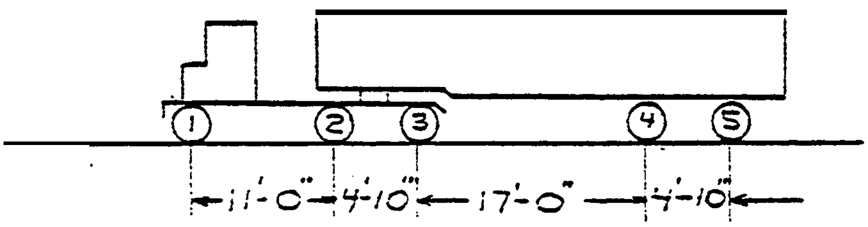
Note that at each axle spacing there are 3 values. The top value (4 tires each axle - 8' wide) generally is used for construction equipment.

1. Check single axles Max = 28,000 lbs

2. Check tandems and tridens
   Example: Tandem spacing = 4'-10"
   Allowable load = 47,075 lbs

3. Check any other grouping where the distance first to last axle of the group equals 18 feet or less.

Example:



Description	Actual	Allowable	Remarks
Axle 1	16000 lbs	28,000 lbs	OK
2 + 3	45000 lbs	47,075 lbs	OK
4 + 5	53000 lbs	47,075 lbs	NG
1 + 2 + 3	61000 lbs	58,625 lbs	NG
2 + 3 + 4	Since L > 18 feet 2 + 3 + 4 does not govern		

The vehicle exceeds purple loading.

PURPLE ON CONSTRUCTION PROJECTS

Section 7-1.02 of the Standard Specification governs extra legal loads for earthmovers, trucks, truck and trailer combinations and the Permit Policy covers other equipment. The two most common classes are as follows:

Concrete Trucks

Concrete trucks which use booster axles travel on the highway and should come on to the project with a load not exceeding legal. The Permit Policy does not allow over legal loads with these trucks. However on the job, discharging or traveling, the booster is up and the truck becomes over legal. Section 7-1.02 of the Standard Specifications allows trucks over legal on bridges with up to 28,000 pounds for single axles and 48,000 pounds for the rear tandem axles. This limits most trucks to hauling a maximum 7 l/2 to 8 cubic yards. These trucks should be weighed to confirm allowable specification loading.

Truck Cranes

Fully equipped truck cranes are permitted to cross bridge structures on construction projects provided they conform to the Permit Policy. Operation of Cranes in a stationary lifting mode on bridge decks are covered elsewhere in this memo. The smaller two and three axle cranes are covered by the general requirements of the Permit Policy.

An exception to this policy is made for truck cranes with axle width, out to out of tires of 120" or more and having 4 or more 14" minimum width tires per axle on the real tandem or triden. These truck cranes may be allowed a bonus or load transfer. Most of the larger three and four axle and all five and six axle cranes fall into this category. The application of this exception can be confusing.

Eight charts are included herein for the determination of purple loads for truck cranes with from two to six axles. Both regular and bonus types (120" wide - 4-14" tires per axle) are included. Truck cranes meeting purple policy should only be allowed on bridges which have a purple capacity rating.

INSTRUCTIONS FOR USING CHARTS FOR DETERMINING IF TRUCK CRANES ARE ALLOWABLE ON PURPLE RATED BRIDGES

1. The Contractor must furnish the following information to the Structure Representative: The truck crane make, model, and a distinctive identifying number; the axle spacing, number and size of tires on each axle; the out to out spacing of the tires, and the axle loads of the fully rigged crane determined with boom both forward and back.

2. The Structure Representative must verify that the information regarding the physical dimensions of the truck crane are correct. He must also verify that the loads as given by the Contractor are reasonably close to those actually imposed on the ground. Weigh if necessary.

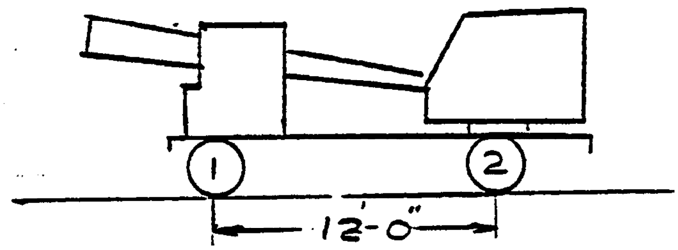
3. The Structure Representative then compares the proposed actual loads with the allowable loads as shown on the appropriate chart. Each chart has two sections. One for distance L greater than 18 feet and one for L equal 18 feet or less. The distance L is shown as a possible combination of axles which may be within 18 feet and which can control the loading. Allowable loads for tandems and tridens use L = 4' -6" and 5'-0" and are shown.

   It is important to check <u>ALL</u> axle combinations on a given crane.

   Note that where a load transfer is indicated up to 7,000 lbs may be transferred. Use only enough to qualify the rear tandem or triden so that the front is not reduced more than necessary. The gross must not exceed the value without transfer in any case.

ALLOWABLE PURPLE LOADS EXAMPLES

2 AXLE CRANE

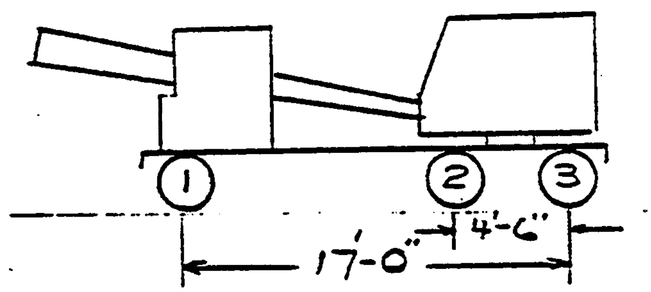


Axle 1	20,000 lbs	Allowable	28,000 lbs	OK
Axle 2	25,000 lbs	Allowable	28,000 lbs	OK
Axle 1 + 2	45,000 lbs	Allowable	54,600 lbs	OK

3 AXLE CRANE

Front Axle has 2 - 12" tires

Rear Axles have 4 - 14" tires and out to out of tires is 122"



Axle 1	20,000 lbs	Allowable	28,000 lbs	OK
Axle 2 + 3	50,000 lbs	Allowable	53,725 lbs	OK
Axle 1 + 2 + 3	70,000 lbS	Allowable	66,850 lbs	NC

Same crane With 12" tires on all wheels

Axle 1	20,000 lbs	Allowable	28,000 lbs	OK
Axle 2 + 3	50,000 1bs	Allowable	36,725 lbs	NC
Axle 1+ 2 + 3	70,000 lbs	Allowable	59,850 lbs	NG

ALLOWABLE PURPLE LOAD EXAMPLES

FOUR AXLE CRANE

The Contractor wants to move a truck crane across completed structures on the contract. He provides the following information about the truck crane.

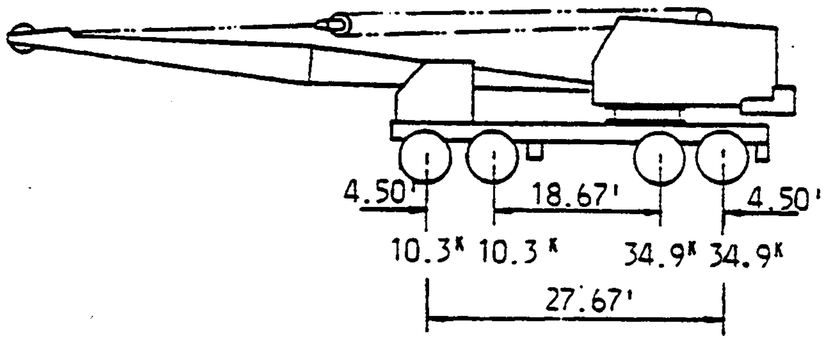
Truck Crane is a 4 axle, **BLAW-WHOOPING,** model EX 1972. Contractor's equipment No. is 1453, Lic. No. 735463.

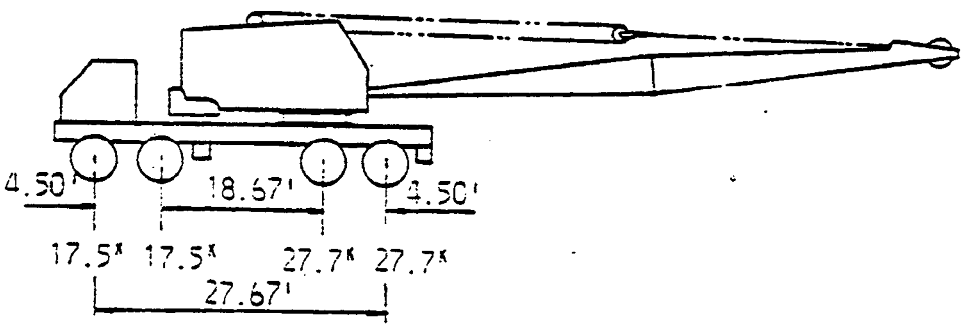
Front Tandem has 2-12" tires per axle and out to out of tires is 123".

Rear Tandem has 4-l4" tires per axle and out to out of tires is 123 inches.

The crane, when weighed, was equipped with a 40'-34" deep angle boom, a boom gantry, complete front and rear out riggers, and complete counterweights.

Axle spacing, axle weights and boom position are as shown below.





Problem Solution

As the truck crane, proposed for use by the Contractor, is a four-axle crane that has more than four 14" tires per axle on the rear tandems, and has an out to out width of tires greater than 120 inches, use Chart "E" (Attachment No. 3, Sheet 10 of 13). Check the truck crane with the boom horizontal in both the forward and rear positions.

Boom to Front - With Load Transfer

Description	Allow. Load	Actual Load	Remarks
Axle 1 + 2	46,725 lb	20,600 lb	OK
Axles 3 + 4	53,725 lb	69,800 lb	NG
Axles 1 +2 + 3	Does not apply since L > 18 feet		
Axles 2 + 3 + 4	Does not apply since L > 28 feet		
Axles 1+ 2 + 3 + 4	93,450 lb	90,400 lb	OK

Conclusion: With boom to front the truck crane exceeds the allowable "purple loading":

Boom to Rear - With Load Transfer

Description	Allow Load	Actual Load	Remarks
Axle 1+ 2	46,725 lb	35,000 lb	OK
Axles 3 + 4	53,725 lb	55,400 lb	NG
Axles l+ 2 + 3	Does not apply since L > 18 feet.		
Axles 2 + 3 + 4	Does not apply since L > 18 feet.		
Axles 1+ 2 + 3 + 4	93,450 lb	90,400 lb	OK

Conclusion: With boom to rear, and considering that a load transfer was allowed, the truck crane exceeds the allowable "purple loading".

Four Axle Crane

Same crane as in previous example except distance between axle 2 and axle 3 is 11"-6".

Now L = ll'-6" plus 4'-6" = 16.0 feet

Boom to Front with Load Transfer

Description	Allow. Load	Actual Load	Remarks
Axle 1+ 2	46,725 lb	20,600 lb	OK
Axle 3 + 4	53,725 lb	69,800 lb	NG
Axle 2 + 3 + 4	65,800 lb	80,100 lb	NC
Axle 1+ 2 + 3	65,800 lb	55,500 lb	OK
Gross	93,450 lb	90,400 lb	OK

Conclusion: With boom to front the truck crane exceeds the allowable purple loading.

Boom to Rear with Load Transfer

Description	Allow. Load	Actual Load	Remarks
Axle 1+ 2	46,725 lb	35,000 lb	OK
Axle 3 + 4	53,725 lb	55,400 lb	NG
Axle 2 + 3 + 4	65,800 lb	72,900 lb	NC
Axle 1+ 2 + 3	65,800 lb	62,700 lb	OK
Gross	93,450 lb	90,400 lb	OK

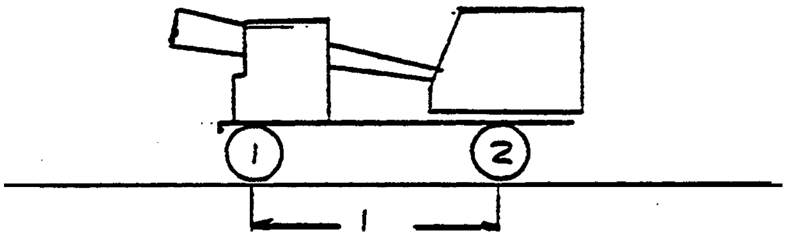
Conclusion: With boom to rear the truck crane exceeds the allowable purple loading.

ALLOWABLE PURPLE LOADS   Chart A

TWO AXLE CRANES

The loads shown below are the maximum permited regardless of tire size, axle width out to out of tires, or numbers of tires per axle.

To qualify purple: EACH of the axles shall NOT exceed the value shown. The allowable gross is a function of "L".



	Max allowable lbs
Axle 1	28,000
Axle 2	28,000

Gross load is governed by the basic permit policy formula,

$$\text{Gross (lbs)} = 2.5 \times 700 (L + 40)$$

		Max Allowable lbs
Gross	L = 8 ft	50,400
	9	51,450
	10	52,500
	11	53,550
	12	54,600
	13	55,650
	13	56,000

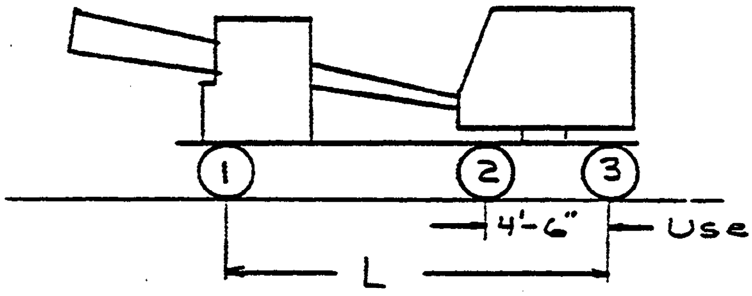
* 56,000 lbs is the maximum for any two axle crane over L=14

ALLOWABLE PURPLE LOADS   Chart B

THREE AXLE CRANES

Axle width less than 120" out to out of tires OR less than 4-14" minimum width tires an each axle of the rear tandem.

TO QUALIFY PURPLE: EACH of the combination weights shown shall NOT be exceeded.



ASSUME

Axle 2 = Axle 3

L MORE THAN 18 FEET

	Max allowable lbs
Axle 1	28,000
Axle 2 + 3	46,725
Gross for any L > 18 feet	74,725

L = 18 FEET OR LESS

		Max allowable lbs
Axle 1		28,000
Axle 2 + 3		46,725
Axle 1 + 2 + 3	L = 14 ft	56,700
	15	57,750
	16	58,800
	17	59,850
	18	66,900

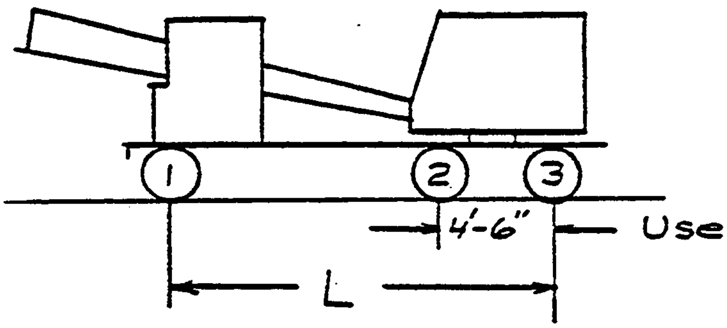
For any other L < 18
Gross = 1.5 x 700 (L + 40)

ALLOWABLE PURPLE LOADS Chart C

THREE AXLE CRANES

Axle width out to out of tires 120" or more and having 4 or more 14" minimum width tires per axle on the rear tandem.

To QUALIFY PURPLE: EACH of the combination weights shown shall NOT be exceeded.



ASSUME

Axle 2 = Axle 3

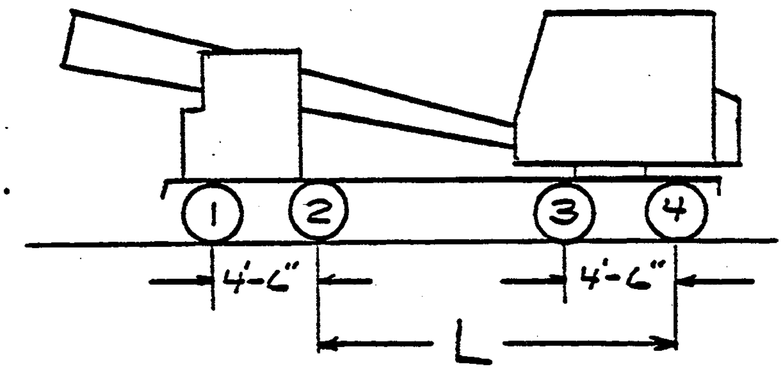
L MORE THAN 18 FEET		Max allowable lbs
Axle 1		28,000
Axle 2 + 3		53,725
Gross for any L > 18 feet		81,725

L = 18 FEET OR LESS		Max allowable lbs
Axle 1		28,000
Axle 2 + 3		53,725
Axle 1 + 2 + 3	**L = 14 ft**	63,700
	15	64,750
	16	65,800
	17	66,850
	18	67,900

FOUR AXLE TRUCK CRANES

Axle width less than 120" out to out of" of fires OR less than 4-14" minimum width tires on each axle of the rear tandem.

TO QUALIFY PURPLE: EACH of the combination weights shown shall NOT be exceeded.



ASSUME

Axle l = Axle 2

Axle 3 = Axle 4

		Max Load Us
L MORE THAN 18 FEET		
Axle 1 + 2		46,725
Axle 3 + 4		46,725
Gross for any L > 18 feet		93,450
L = 18 OR LESS		
Axle l + 2		46,725
Axle 3 + 4		46,725
Axle 2 + 3 + 4	L = 14 ft	56,700 **
	15	57,750 **
	16	58,800 **
	17	59,850 **
	18	60,900 **
Axle 1 + 2 + 3	Same as Axle 2 + 3 + 4	

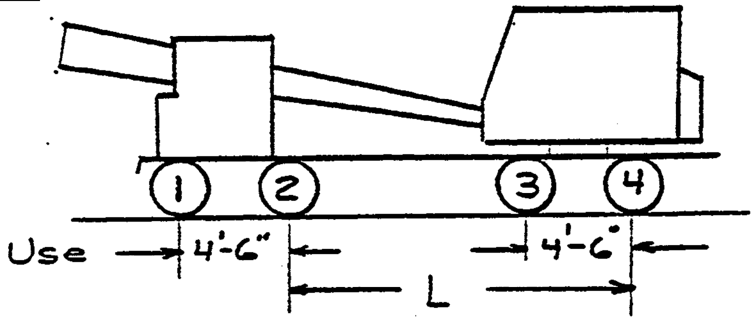
** These values will limit the maximum allowable above on axles 1+2 or axles 3+4. For a given L, allowable gross vehicle weight will vary depending on relative actual weights on front and rear tandems. If the above combinations are not exceeded the allowable gross will be satisfactory.

ALLOWABLE PURPLE LOADS   Chart E

FOUR AXLE TRUCK CRANES

Axle width out to out of tires 120" or more and having 4 or more 14" minimum width tires per axle on the rear tandem.

TO QUALIFY PURPLE: Each of the combination weights shown shall NOT be exceeded.



ASSUME
Axle 1 = Axle 2
Axle 3 = Axle 4

L MORE THAN 18 FEET

	Max allowable range (lbs)	
Axle 1 + 2	46,725	39,725 *
Axle 3 + 4	46,725 or	53,725 *
Gross	93,450	93,450

* Up to 7000 lbs may be transferred from front to rear tandem i.e. Front tandem allowable may be 39,725 with a maximum of 33,725 on rear tandem. Gross maximum remains 93450.

L = 18 FEET OR LESS

		Max allowable lbs	
Axle 1 + 2		46,725	
Axle 3 + 4		53,725	
Axle 2 + 3 + 4	L = 14 ft.	63,700	**
	15 ft.	64,750	**
	16 ft.	65,800	**
	17 ft.	66,850	**
	18 ft.	67,900	**
Axle 1 + 2 + 3	Same as Axle 2 + 3 + 4		

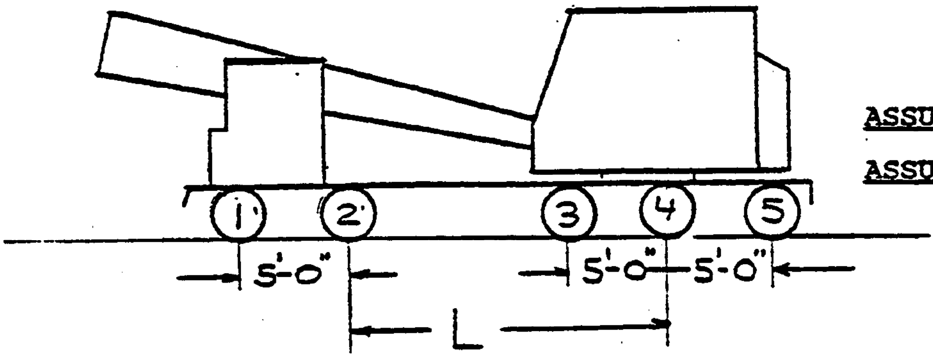
** These values will limit the maximum allowable above on axles 1+2 or axles 3+4. For a given L, allowable gross vehicle weight wil1 vary depending on relative actual weights on front and rear tandem. If the above combinations are not exceeded me allowable gross will be satisfactory.

ALLOWABLE PURPLE LOADS **Chart F**

FIVE AXLE TRUCK CRANES - 2 AXLE STEERING

Axle width out to out of tires 120" or more and having 4 or more 14" minimum width tires per axle on the rear triden.

TO QUALIFY PURPLE: EA<u>CH</u> of the combination weights shown shall N<u>OT</u> be exceeded.



ASSUME Axle 1 = 2

ASSUME Axle 3 = 4 = 5

L = 15 FEET OR MORE	**Max allowable Range (lbs)**	
Axle 1 + 2	47,250	40,250 *
Axle 2 + 3 + 4	52,500 OR	59,500 *
Gross	99,750	99,750

* Up to 7000 lbs may be transferred from front tandem to rear triden providing <u>gross maximum remains</u> at <u>99,750.</u>

L LESS THAN 15 FEET		Max allowable lbs	
Axle 1 + 2		47,250	
Axle 3 + 4 + 5		59,500	
Axle 1 + 2 + 3	L = 11 ft	60,550	**
	12	61,600	**
	13	62,650	**
	14	63,700	**
	15	64,750	**

Axle 1 + 2 + 3 does not control above L = 15'-0"
Axle 2 + 3 + 4 dues not control above L = 10'-3"

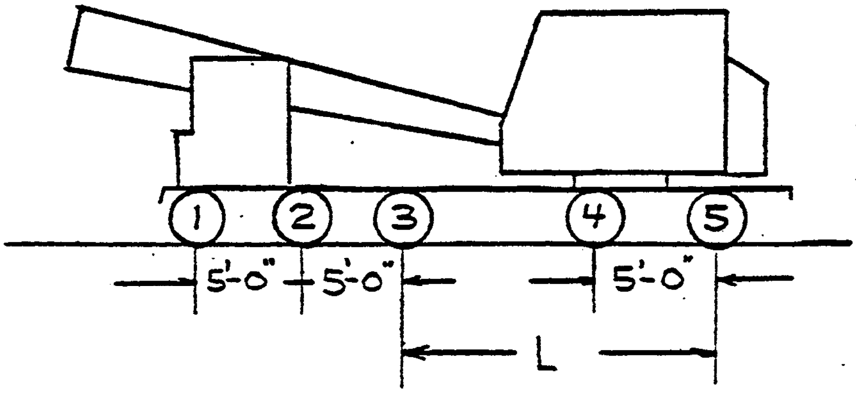
**** These values may limit the maximum allowable above on axle 1 + 2 and/or axle 3 + 4 + 5. For a given L, allowable gross vehicle weight will vary depending on relative actual weights on front tandem or rear triden. If the above combinations are not exceeded the allowable gross will be satisfactory.**

ALLOWABLE PURPLE LOADS   Chart G

FIVE AXLE TRUCK CRANES - 3 AXLE STEERING

Axle width out to out of tires 120" or more and having 4 or more 14" minimum width tires per axle on the rear tandem.

TO QUALIFY PURPLE: EACH of the combination weights shown shall NOT be exceeded.



ASSUME Axle 1 = 2 = 3

ASSUME Axle 3 = 4

L MORE THAN 18 FEET

	Max allowable (lbs)			
Axle 1 + 2 + 3	52,500		45,500	*
Axle 4 + 5	47,250	OR	54,250	*
Gross	99,750		99,750	

* Up to 7000 lbs may be transferred from front triden to rear tandem providing gross maximum remains at 99,750.

L = 18 FEET OR LESS

		Max allowable lbs	
Axle 1 + 2 + 3		52,500	*
Axle 4 + 5		54,250	
Axle 3 + 4 + 5	L = 15 ft	64,750	**
	16	65,800	**
	17	66,850	**
	18	67,900	**

Axle 2 + 3 + 4 does not control above L = 9'-2"

** These values may limit the maximum allowable above on axle 1 + 2 + 3 and/or axle 4 + 5. For a given L, allowable gross vehicle weight will vary depending on relative actual weights, on front triden or rear tandem. If the above combinations are not exceeded the allowable gross will be satisfactory.

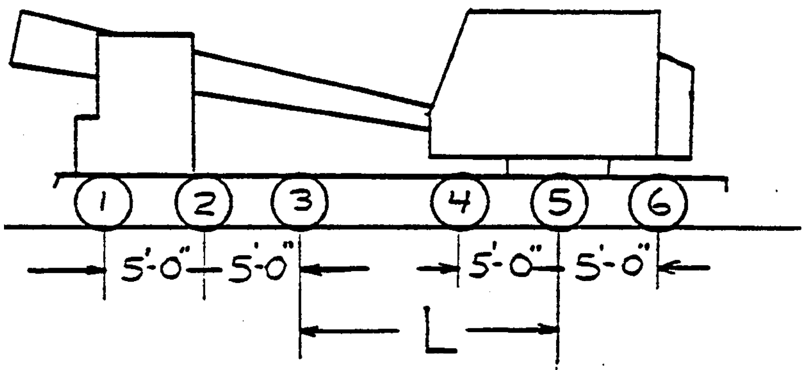
ALLOWABLE PURPLE LOADS

Chart H

SIX AXLE TRUCK CRANES

Axle width out to out of tires 120" or more and having 4 or more 14" minimum width tires per axle on the rear triden.

TO QUALIFY PURPLE: EACH of the Combination weights shown shall NOT be exceeded. Six axle truck cranes currently permitted in California can not make purple weight with the upper works in place. They often do not qualify with just the carrier.



ASSUME

Axle 1 = 2 = 3

Axle 4 = 5 = 6

L MORE THAN 18 FEET

	Max allowable	Range (lbs)
Axle 1 + 2 + 3	52,500	45,500 *
Axle 2 + 4 + 5	52,500 OR	59,500 *
Gross	105,000	105,000

* Up to 7000 lbs may be transferred from front triden to rear triden providing gross maximum remains at 105,000.

L = 18 FEET OR LESS

		Max allowable lbs	
Axle 1 + 2 + 3		52,500	
Axle 4 + 5 + 6		59,500	
Axle 3 + 4 + 5	L = 15 ft	64,750	**
	16	69,800	**
	17	66,850	**
	18	67,900	**

Axle 2 + 3 + 4 does not control

** These values may limit the maximum allowable above on axle 1 + 2 + 3 and/or axle 4 + 5 + 6. For a given L, allowable gross vehicle weight will vary depending on relative actual weights on front triden or rear triden. If the above combinations are not exceeded the allowable gross will be satisfactory.

FORM HC-5
REV. 1-761

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

CONTRACT CHANGE ORDER NO.__________ SUPPL. NO._____

ROAD____________________ SHEET_____ OF_____ SHEETS

FEDERAL NO.(S)____________________ CONTRACT NO.__________

To____________________ Contractor
You are hereby directed to make the herein described changes from the plans and specifications or do the following described work not included in the plans and specifications on this contract.

NOTE: This change order is not effective until approved by The Chief Engineer.

Description of work to be done, estimate of quantities, and prices to be paid. Segregate between additional work at contract price, agreed price and force account. Unless otherwise stated, rates for rental of equipment cover only such time as equipment is actually used and no allowance will be made for idle time.

Change requested by

The last percentage shown is the net accumulated increase or decrease from the original quantity in the Engineer's Estimate.

EXAMPLE CHANGE ORDER TO AUTHORIZE BRIDGE REDESIGN

As provided in Section 7-1.02 of the Standard Specifications, modify substructure of the Van Koevering Avenue Undercrossing, Bridge 54-1001, as shown on Sheets 2 and 3 of this change order to accommodate construction overloads.

It is agreed that the contractor will furnish all labor, equipment and material and perform all work required to accomplish the structural alterations shown on Sheets 2 and 3 at no cost to the State.

It is further agreed that the State will deduct from such progress and final pay estimates as may became due under the contract the actual cost for redesigning this Van Koevering Avenue Undercrossing to accommodate construction overloads, 'subject to a maximum deduction of $2,000.

Estimated Cost Decrease $ 2,000 or Increase $__________

By reason of this order the time of completion will be adjusted as follows: No adjustment

Submitted by:____________________ Date__________

Approval Recommended:____________________ Date__________

Approved: Chief Engineer by____________________ Date__________

We, the undersigned contractor, have given careful consideration to the change proposed and hereby agree, if this proposal is approved, that we will provide all equipment, furnish all materials, except as may otherwise be noted above, and perform all services necessary for the work above specified, and will accept as full payment therefor the prices shown above.

Accepted, Date__________ Contractor____________________

By____________________ Title__________

If the Contractor does not sign acceptance of this order, his attention is directed to the requirements of the specifications as to proceeding with the ordered work and filing a written protest within the time therein sp

EXAMPLE LETTER TO AUTHORIZE THE USE OF EARTHMOVING EQUIPMENT

To Contractor

Gentlemen:

Your request dated ______(date)________ for permission to cross the______name of bridge)______________ with construction overloads is approved in accordance with the provisions of Section 7-1.02 of the Standard Specifications, subject to the following conditions:

1. The approaches at each end of the bridge shall be completed to the grade required to provide a smooth transition to the bridge roadway, and shall be maintained in a smooth and uniform condition at all times while construction equipment is in use, for a length of not less that 150 feet measured from the bridge ends. Local depressions in the approaches in the vicinity of the bridge ends will not be permitted.

2. Construction equipment, either loaded or unloaded, shall be operated at all times at a speed and in a manner so that jouncing or bouncing of the equipment will not occur while the equipment is crossing the bridge.

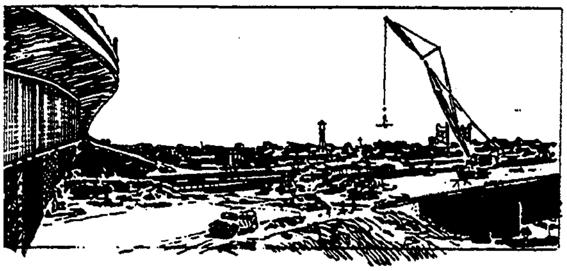
3. Construction equipment shall be confined to the construction equipment land by means of substantial, temporary physical barriers.

4. Only one construction overload will be permitted on the bridge at any one time.

5. On completion of the operation which requires the use of a construction overload, the bridge roadway shall be cleaned and physical barriers used in connection with the construction equipment lane shall be removed and disposed of away from the job site.

Note: Other conditions or restrictions may be added as necessary to suit particular job circumstances.

/s/ Resident Engineer



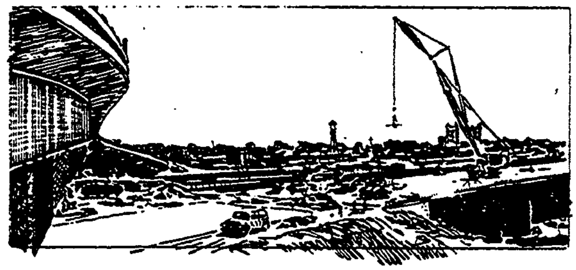
Volume II

TABLE OF CONTENTS-FOR SECTION NO. 155

Memo No.	Date	Title
155-1.0	04-02-90	CLEANING AND PAINTING OF STRUCTURAL STEEL
155-2.0	11-26-84	BRIDGE PAINTING -- ESTIMATING WORK DONE
155-3.0	11-26-84	SAMPLING AND TESTING PAINT

A. P. Bezzone

A. P. BEZZONE, Chief
Office of Structure Construction



CLEANING AND PAINTING OF STRUCTURAL STEEL

General Information

The cleaning and painting of structural steel bridges is a vital, specialized, and often controversial phase of bridge construction and maintenance.

Structure Representatives are responsible for the satisfactory completion of cleaning and painting work in accordance with the contract specifications, Attachment No. 3 to this Bridge Construction Memo is a check list which may be used to aid the Structure Representative in obtaining a satisfactory painting project.

The following information on cleaning and painting methods, procedures and precautions, paint material, inspection techniques and record keeping is intended to provide the Structure Representative's and their assistants with a rudimentary knowledge of the cleaning and painting work. Of course, any specific instructions in the contract specifications will supersede or modify these instructions.

Any special problems with regard to cleaning and painting, which cannot be solved by the Structure Representative, should be referred to Office of Structure Construction.

Purpose of Painting

The paint on structural steel may be described as a relatively impervious barrier imposed between the steel surface and its environment. Paint retards the corrosion of the steel. Corrosion may manifest itself in many forms, and it may have many causes, but the effect is always the same: metal is consumed or deteriorated.

Paint, then, may be considered a low-cost renewable or repairable shield or membrane which is sacrificed to the elements to protect the metal. The service life expectancy of paint coats in California as affected by climatic conditions, is illustrated by the chart shown on Attachment No. 1 to this Bridge Construction-Memo.

The service life of a paint coat is also a function of the quality of the paint coat. Paint must be properly formulated and prepared from ingredients having certain necessary qualities. It must be properly applied to clean surfaces of steel, and the completed film must have adequate thickness. Shortcomings in any of these requirements result in a decreased service life of the paint coat. In California, atmospheric conditions affecting the service life of paint coats vary between two extremes: the saline humidity of the sea coast and the hot aridity of the desert. Between these two extremes are regions where milder weather conditions prevail. Obviously, the need for protection is considerably less under mild exposures than it is under severe exposures. The paint system specified is therefore designed to meet the needs of the area, and conform to the latest pollution regulations imposed on solvent content of paint materials.

Current paint systems consist of either phenolic, or water-borne undercoats and water-borne finish coats. The thicknesses required varies according to the corrosion potential at the site.

Due to air pollution regulations, a paint system consisting of water-borne primers and top coats has been developed. This system consists of 4 mils of undercoat applied in 2 or more applications and 4 mils of finish paint applied in 2 or more applications. No vinyl wash primer is used.

Water-borne paints generally require higher temperatures and lower relative humidities than some other paints to dry properly. Care should be taken not to permit painting when the atmospheric or surface temperature is at or below 50 F, or when the relative humidity exceeds 75 percent. Temperature and relative humidity should remain within the above limits for approximately 4 hours after application to permit adequate drying.

From past experience with paint systems; the specification of multiple coat applications and the minimum dry film thicknesses of paint coats have evolved. Most paints used on structural steel. contain varying amounts of volatile solvents which, when they evaporate during the drying process, leave minute holes in the paint film. The application of multiple coats of paint, not too thin or too thick, tends to overcome the adverse pin-hole pattern in each coat and assures a truly impervious membrane. On any particular job, the specification of paint coat thickness of either paint system is adapted to the prevailing exposure conditions.

Most paints will not tolerate extra thick applications or puddles. If too much paint is applied, or puddles of the material are left on the surface, the coating will crack and lose bond with the steel or underlying coat. Each application should be held to near the amount specified.

Surface Preparation

The most important factor affecting the protective service life of a paint is the surface preparation prior to painting. The best paint available will not give optimum service when applied over improperly cleaned surfaces. It is essential, therefore, that paint is applied only to clean, sound, dry surfaces.

Although several methods of surface preparation are employed in the painting industry, it has been found that blast-cleaning and steam-cleaning are the most effective and least expensive methods. These two methods are specified almost exclusively. Occasionally, in mild exposure areas or where the type and amount of rust does not warrant the expense of blast-cleaning, hand cleaning methods may be specified.

Blast-cleaning is frequently referred to as sandblasting. However, since the abrasive used need not be limited to sand, the Office of Structure Construction has adopted the less restrictive term, blast-cleaning, in its specifications.

Blast-cleaning is simply the propulsion of an abrasive against an object, and the cleaning is accomplished by abrasive action. Various sources of power may be used to propel the abrasive, but the one most commonly used is compressed air. Another source is centrifugal force as used in large machines designed for the purpose. These machines are used only in shop installations because of their size and immobility. In field work, compressed air seems likely to remain the chief power source for some time. When dictated by adverse environmental impact, wet blast-cleaning may be specified. The power source for this method is either high pressure water or steam.

Sand, because of its abundance and consequent low cost, is the principal abrasive used. The only requirement imposed in the specifications is that the material be clean, dry, of proper grading, and meet requirements of the Air Resources Board for "Dry Unconfined Blasting". The degree of hardness is not specified.

Sand, obtained from commercial sources generally meets our requirements. Use of unwashed beach or river sand is not permitted because contaminants or too many fines are often present, It does not meet ARB requirements.

Other abrasives used on a lesser scale are steel shot, steel grit and slag from copper, nickel, and silver smelting processes. The use of steel shot or steel grit is usually limited to shop blasting where recovery for reuse is possible. High initial cost and lack of a practical recovery method prohibit the use of these abrasives in the field.

In the 1988 Standard Specifications "Blast-Cleaning," has been described in a different manner. This description is not intended to lower the degree of cleanliness of the steel from past years, but conforms more closely to language used in outside Industry.

Steam-cleaning consists of washing the surface to be painted with steam in which a biodegradable detergent soap has been incorporated in the feed water, or applied directly to the surface to be cleaned. The steam is directed against the surface, and the contaminants, loosened by the detergent, are carried away by the flushing action of the condensed steam. Any residue remaining on steam-cleaned surfaces should be flushed with fresh water before painting.

Steam-cleaning is used principally in maintenance work when spot-cleaning and painting are specified, although it may occasionally be used on shop-coated steel-in new construction work if the surfaces have become contaminated by dust, oil or other contaminating products. The primary purpose of steam-cleaning is to remove surface contaminants which would impair bonding of new paint to existing coatings. Steam-cleaning will not remove rust, and if rust is present after steam-cleaning, the operation will generally be followed by spot blast-cleaning.

An interval of at least 24 hours should elapse after steam cleaning before paint is applied.

A steam-cleaning supplement which describes the operation, equipment used and the detergent intermingling procedure in more detail is available upon request from the Office of Structure Construction.

Paint Application

The paint coats specified generally consist of one or more undercoats. The various coats or layers are planned and specified (1) to achieve an impervious membrane which inhibits corrosion: (2) to protect the steel against impact or abrasion and (3) to give the structure a pleasing appearance.

The normal functions of undercoats are to inhibit corrosion, to provide a suitable base for the finishing coats and to present a secondary barrier to any moisture penetrating the finishing coats.

Finishing coats comprise the tough outer layer of the paint film which is directly exposed to the weather. They are the weathering or wearing coats of a paint system and must, therefore, have a harder, more impervious surface than the undercoats. Two applications of finishing coat paint are normally specified.

Paint may be applied to structural steel by brush, roller or spray, but regardless of the method used, care must be exercised in the application in order that the maximum service-life may be realized. It is the responsibility of the Structure Representative to see that the paint is applied properly. The paint should be well mixed and uniformly applied, and any skips or holidays should be picked up before subsequent applications are allowed, since the smallest break or thin spot in the paint film is a potential trouble spot.

All formulations now in use, EXCEPT the inorganic zincs can be applied by any of the previously mentioned methods. Spraying is the only satisfactory method for application of inorganic zincs to large surfaces. However, small holidays or skips which sometimes occur around rivets or bolts can be picked up with a brush, and areas inaccessible with a spray gun should be swabbed or brushed.

Experience has taught us that "Airless" spray is inferior to conventional spray, on most bridge structures, due to lack of control of the amount of paint material being dispersed from the nozzle.

Paints for use on structural steel, except inorganic zinc primer, are manufactured ready for-application and thinning is not necessary, nor should it be tolerated. Inorganic zinc primer may be thinned as recommended by the manufacturer.

Painting for appearance may be considered of secondary importance to painting for protection, but it is evident that the public is aware of bridge appearance. Both maximum protection and pleasing appearance can be achieved by a paint job properly done. The most common causes of poor appearance are runs or sags in the paint film and paint spray or splatters on the concrete portion of the structure. By using care and precaution, it is far easier to prevent-these defects than it is to correct them.

Thickness of Paint Film

Dry film thickness of the paint film is always specified in either the special provisions or, by reference, in the Standard Specifications. In all cases, the specified mil thickness is the minimum on all surfaces and does not mean the overall averages.

Paint dry film thickness is measured by a magnetic flux gauge called "E1cometer" or "Positector" Gauges are supplied by the Office of Structure Construction with instructions for their use. These devices are delicate and expensive instruments and should, therefore, be handled with care. Gauges should not be stored near active electrical circuits, and they should not remain near welding equipment longer than absolutely necessary. Periodic checks to determine the accuracy of the gauge is necessary: these checks may be made by using the shims provided. It is not the intention of the Office of Structure Construction to penalize a Contractor by requiring more thickness than specified, but, on the other hand, we should be sure that we do not get less. All measurements should be taken with the gauge placed firmly at right angles to the area being measured, even a slight slanting of the device gives a high reading, as will lack of solid contact. Recalibrate gauges on different types and sizes of steel. Reading differences have been noted between webs, stiffners and braces.

It is often necessary on small jobs or near the completion of large ones to measure a film thickness of paint which is not hard enough to prevent indentation by the film-thickness gauge. If a close inspection shows such a condition, the reading is certain to show less thickness than is actually on the steel. Correction can be made by placing a shim between the paint film and the film thickness, gauge and deducting the thickness shown on the shim from the reading taken.

The importance of adequate paint film thickness cannot be overstressed. All other things being equal, it is one of the factors that determines the service life of a paint job. It follows, therefore, that sufficient measurements should be taken to assure specified thicknesses in all places.

Protective Measures

Inherent in a bridge painting operation is the possibility of the creation of a nuisance or of the physical damage to adjacent property or to the traveling public. This is particularly true on contracts involving the repainting of structures under traffic.

Although the responsibility for the prevention of damage rests with the Contractor, the Structure Representative must constantly be aware of the job situation and should not hesitate to call the existence of hazards or potential sources of damage to the Contractor's attention.

In the event passing automobiles are spattered with paint, little damage will occur if the paint is immediately removed with mineral spirits or with water for water-borne paints. However, this should not be a common occurrence. A prudent Contractor will use drop cloths, screens, overhead tarps, and the like to adequately protect passing traffic or adjacent property.

Particular emphasis should be placed on the protection of concrete surfaces which are a part of the structure. The Contractor should not be allowed to mix paint or charge paint pots on bridge decks without adequate drop cloths. It is next to impossibleto remove paint-from concrete, and particular care should be exercised to prevent spattering such surfaces. Thinners and paint removers should not be used in attempting to remove paint from rough concrete surf aces. After the paint is dry, the area should be rubbed with a stone and wire brushed, or lightly blast-cleaned.

Paint which is being sprayed can drift as much as a quarter mile or more, and Contractors should be reminded of this possibility, particularly if automobiles are being parked nearby.

In general, the best protective measure is the anticipation of possible damage and prevention of its occurrence.

Paint Records and Reports

The Office of Structure Construction has developed a series of special record forms for use in keeping daily job records on each phase of the cleaning and painting operation. These forms will be furnished to the Structure Representative at the beginning of his assignment to a particular project, Samples of these forms (DH-OS M5, DH-OS M8, DH-OS M11, and DH-OS M78) are included in Section 16 of the Bridge Construction Records and Procedures.

Paint record sheets were developed to simplify the reporting of statistical data as well as to ensure uniformity in record keeping. Structure Representatives should be familiar with the use of the sheets and should enter the required information in accordance with procedures recommended by the painting section.

In addition to the paint records. The Resident Engineer's and/or Assistant Resident Engineer's Daily Reports (HC-1O and HC-1OA) are required for the painting operation.

The Blast-cleaning and paint record form (DH-OS M8, Daily clean and Paint Record), is a diary form used by the Structure Representative for the various phases of the cleaning and painting work. These diaries have the same significance as the general diary forms HC-10 and HC-1OA and should, therefore, receive the same degree of care in their preparation and distribution.

Form DH-OS M78, is a record of spot-blast-cleaning performed. The purpose of this form is to have the Structure Representative and the Contractor's representative agree, on a daily basis, on the amount of spot-blast-cleaning performed.

On repainting projects, the Structure Representative will prepare, from the information gathered in the daily diaries, cost data for the various phases of blast-cleaning and painting. This data will be entered on the paint data sheets, Form DH-OS M5, Clean and Paint Cost Summary. Use of this form aids the Structure Representative in making a systematic and uniform record of cost data.

Following completion of the painting operation, statistical information included on the paint record sheets is summarized on a special summary sheet, Form DH-OS M11, Paint Record. The primary purpose of the information summarized on this form is to provide a sound basis for estimating the cost of future painting projects.

It also provides information regarding the type and quantities of paint used, which information will be valuable when negotiation with pollution agencies in regards to removing paint and repainting the structure on future painting projects. Therefore, the form must be carefully and accurately completed if it is to have any real value.

The original Form DH-OS M11 and supporting paint record forms are retained in the job files until project completion, at which time they are submitted to the Sacramento Office with the Report of Completion. One copy of Form DH-OS M11 should be sent to the Office of Structure Construction as soon as the painting is completed.

Surface Area Computations

The area to be painted is an important part of the paint inspection procedure the surface area must be known to enable the Structure Representative to determine the true rate of progress and to calculate coverage rates. Surface area calculations are also of great value in the planning of future painting contracts. Surface areas of most structures are available in the Sacramento Office of Structures Maintenance. If they are not available, it will be the responsibility of the Structure Representative assigned to the project to calculate them. All calculations should be clearly shown so they may be easily checked by another person. Include subtotals for each span and separate summary sheets for each structure in the project.

Surface area computations will be submitted with Form DH-OS M11 as an attachment to the Report of Completion.

On request, charts to assist in the calculations of surface areas will be furnished to the project by the Office of Structure Maintenance.

Standard Paints Used by the Office of Structure Construction

The different types of paint currently being used by the Office of structure Construction are identified on Attachments No. 2 and 3 of this Bridge Construction Memo. Attachment No. 2 is a list of the standard paints used by the Office of structure Construction and Attachment No. 3 is a working list of approved water-borne inorganic zinc rich primers. No paint brand shall be used unless it is on the Departments current list of approved paints or meets the specifications of the Departments standard paints. In order to give a complete listing of allpaints which may possibly be specified in Office of Structure Construction work, we have included the specifications and descriptions of wood and concrete paints in the tabulation, although the painting of wood or concrete is not discussed in this memo.

Environmental Protection

It is the intention of the Office of Structure Construction to comply with regulations imposed by various public environmental protection agencies. These enforcement agencies are now operative in most areas of the state. Their primary concern is air and water pollution as well as noise abatement.

In order to comply with present and foreseen regulations, new cleaning and painting procedures have been and are being developed. In general, dust created by blast-cleaning, ground pollution from old lead and zinc paints, and overspray from paints are the chief offenders to the environment.

Curtailment methods for dust and waste products include confinement within the immediate work area and use of abrasives which create less dust. Wet-blast cleaning may be another alternative, subject to approval of the engineer.

Confinement of waste products and dust is accomplished by using water curtains, planking, or by draping tarps, potato sacking, heavy-duty polyethylene bags or sheets, or similar materials around and under the work space. The confined waste materials are then collected and hauled to an approved dump site by an authorized transporter.

Copper, silver and nickel slags are sources of abrasives now in fairly common use. These abrasives are more expensive than sand. All abrasives, for dry, unconfined blasting including sand, must be approved by the Air Resources Board.

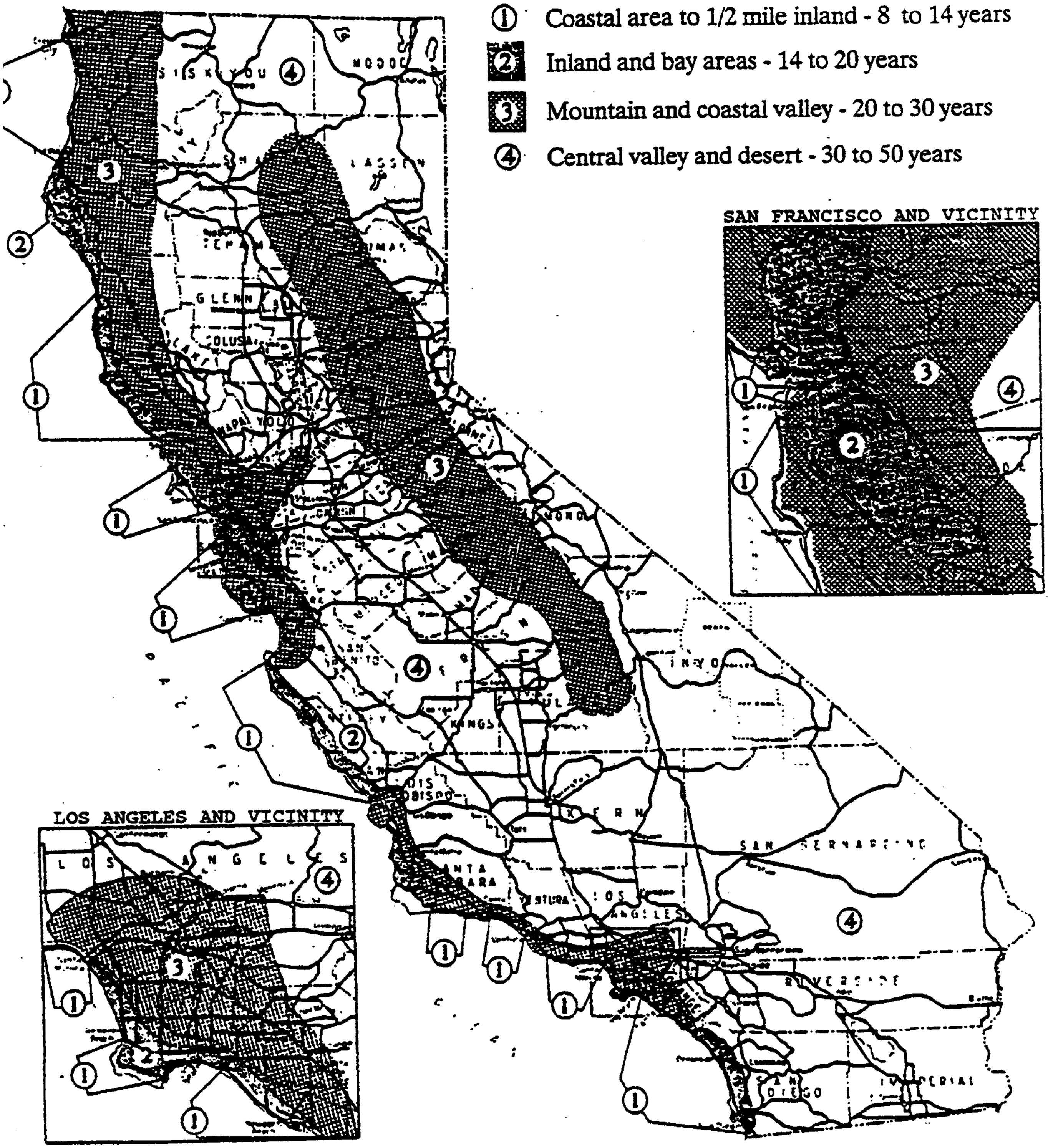
Wet-blast cleaning when it is specified may be done by two methods. One method is the so-called "ring" method. It consists of a perforated ring attached to the blasting nozzle. Water mist forced through holes in the ring mixes with the abrasive at the nozzle and effectively inhibits dust. However, rusting starts immediately on -freshly blasted metal, and the method is impractical for that reason. It is an effective method to use when blast-cleaning concrete, stucco or wood. The other method uses high pressure water or steam as the abrasive impellent. A dilute solution of sodium nitrite added to the water or steam inhibits rusting until the prime coat can be applied. The later method can be used, when necessary, provided excess water can be controlled. Both methods are more costly than dry blast-cleaning and will not be specified unless necessary to meet environmental regulations.

Lead pigmented paints are no longer being specified-for use on structural steel because of their toxicity.

PAINT SERVICE LIFE ON STRUCTURAL STEEL BRIDGES

Legend

1. Coastal area to 1/2 mile inland - 8 to 14 years
2. Inland and bay areas - 14 to 20 years
3. Mountain and coastal valley - 20 to 30 years
4. Central valley and desert - 30 to 50 years



Information shown is approximate only, compiled from records of the Division of Highways for existing bridges on the State Highway system.

STANDARD PAINTS USED BY OFFICE OF STRUCTURE CONSTRUCTION

Phenolic Paints

Spec. No.	Name	Function	Used on
PB-201	Red Primer, High Solids Phenolic Type	Primer	Steel
PB-202	Pink Primer, High Solids Phenolic Type	Primer	Steel
PB-199	Aluminum Phenolic Tung Oil	Finish Paint	Steel

Water-Borne Paints

Spec. No.	Name	Function	Used on
PWB-142	Red Water-borne,	Primer	Steel
PWB-143	Pink Water-borne,	Primer	Steel
PWB-145	Red Water-borne,	Primer	Steel
PWB-146	Pink Water-borne,	Primer	Steel
PWB-87	Flat Gray Water-borne	Finish Paint	Steel
PWB-88	Light Tan Water-borne	Finish Paint	Steel
PWB-89	Tan Water-borne	Finish Paint	Steel
PWB-151	Aluminum, Leafing or Nonleafing Water-borne	Finish Paint	Steel
PWB-82	Light Green, Water-born	Weathering Coat	Steel
PWB-83	Green, Water-borne,	Weathering Coat	Steel
PWB-86	White Tintable, Water-borne	Weathering Coat	Steel
TT-P-19	Acrylic Emulsion - Tintable	Weathering Coat	Masonry
Fed. Spec. TT-P-001984	Wood Primer	Primer & Undercoat	Wood
Fed. Spec. TT-P-96D	White Wood Finish Coat	Weathering Coat	Wood

CALIFORNIA DEPARTMENT OF TRANSPORTATION

QUALIFIED PRODUCTS LIST

WATERBORNE INORGANIC ZINC RICH PRIMER

The following products have been evaluated and determined to provide a material meeting specification requirements for a waterborne inorganic zinc rich primer used in undercoating properly prepared structural steel in transportation maintenance and construction projects.

INORGANIC COATINGS INC.
IC 531 INORGANIC ZINC RICH PRIMER
(800) 345-0531

VALSPAR CORP.
MZ-6 HI-RATIO INORGANIC ZINC RICH PRIMER
(818) 334-8251

DEVOE COATINGS CO.
CATHACOTE 309 WATER BASED INORGANIC ZINC COATING
(504) 272-2470

DU PONT COATINGS CO.
GANICIN 347WB WATER BASED INORGANIC ZINC
(800) 346-4748

The effective period for this list is indeterminate. Other products will be considered for inclusion on this list subject to evaluation and approval by:

California Department of Transportation
Office of Transportation Materials and Research
5900 Folsom Boulevard
Sacramento, CA 95819

CHECK LIST FOR BRIDGE PAINTING PROJECTS

1. Check to see that steam cleaning is doing a satisfactory job of removing all dirt, grease, loose chalky paint, or other foreign materials.

2. Check to see that the specified biodegradable detergent is being used.

3. Check spot-blasting to be assured that all rust has been removed.

4. Check 100% blasted areas to be assured that all rust and old paint has been removed.

5. For "Spot Jobs", check to see that the air pressure and nozzle size meet the specifications.

6. Measure and record spot blast areas daily.

7. Check to see that the first coat of paint is being applied daily. If not, be sure that areas are reblasted before paint is applied.

8. Visually inspect backsides of rivets, tops of diaphragms, tops of bottom flanges, and other hard to reach areas to be assured that they are properly cleaned, and have the required paint coverage.

9. Check to be sure that the access to the work is adequate and that work areas are safe.

10. Observe mixing of paint materials to be assured that the mixing is being-properly done.

11. Require the Contractor to provide safe access to the work so that it can be properly inspected.

12. Check the temperature and humidity at intervals as required to be assured of specification compliance.

13. Check structural steel to be assured that it is dry when paint is applied.

14. Check undercoat for proper thickness before permitting the application of finish coats.

15. Record the quantities of abrasives and paint materials used daily. Also record man hours and hours of equipment use daily. This information is required for the Final Report

16. Enforce the specification requirements concerning the containment of fall-out materials.

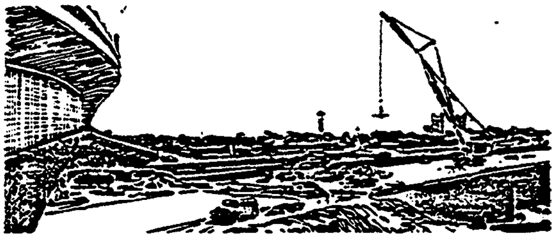
17. Enforce the specification requirements concerning the disposal of used sand and old paint.

18. Check to see that the Contractor is properly protecting deck soffit concrete, concrete caps, concrete piers and other concrete from overspray paint. Areas not so protected must be cleaned before the project is accepted.

19. Check-to see that the Contractor is taking proper precautions to prevent damage to adjacent trees, rocks, and property improvements.

20. Check to see that the Contractor is complying with the OSHA safety requirements.

21. Check to see that waste materials are collected and disposed of properly.



BRIDGE CONSTRUCTION MEMO 155-2.0

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BRIDGE PAINTING -- ESTIMATING WORK DONE

The attached summary sheet (Attachment #l) shows the percentage of the total work included in each phase of a typical bridge painting operation for paint systems used by the Office of Structure Construction.

Although the information shown is approximate only, it is sufficiently accurate for estimating purposes and may be used when computing amounts due on progress pay estimates for work performed under lump sum items.

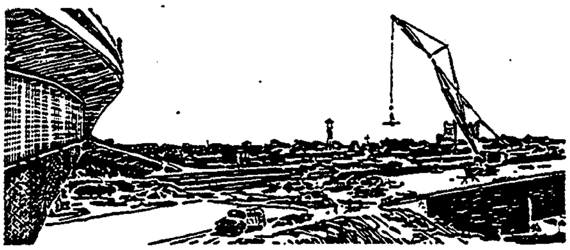
SUMMARY SHEET
PERCENT OF WORK IN LUMP SUM ITEMS

Shop Blast with Inorganic Zinc Water-borne Finish

Operation	Percent	Cumulative Percent
Blast Clean	40	40
Shop undercoats	30	70
Spot clean and undercoats in field	12	82
First finish coat	9	91
Final finish coat	9	100

100% Repaint with Water-borne Paint

Blast clean	60	60
First undercoat	10	70
Second undercoat	10	80
First finish coat	10	90
Second finish coat	10	100



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SAMPLING AND TESTING PAINT

The following sampling procedures are to be followed in order to be assured that all paint which is to be applied to permanent portions of the work complies with the Contract Specifications.

After the Contractor places an order for paint with a manufacturer, the manufacturer must notify the Transportation Laboratory of his intent to manufacture and package the paint. Transportation Laboratory personnel will then visit the manufacturing facility and take samples of the batch of paint. Depending on the manufacturer, and the type of paint, it may either be sampled before or after packaging. In either case the paint is identified by the manufacturers batch number and the date of manufacture. At the time of sampling, the State Inspector assigns a State lot number to the batch which then becomes a part of the identification.

When the testing is completed, the Transportation Laboratory notifies the manufacturer of the test results. If it is confirmed that the batch meets specifications, the manufacturer can then package and label the paint, or just label it if the paint had been packaged prior to taking the sample, The labeling should be in accordance with Section 91-1.03 of the Standard Specifications which states that "All containers of-paint shall be labeled showing the exact title of the paint specification, State specification number, manufacturer's name, date of manufacture, State lot number, and manufacturer's batch number". In addition to this, the State Inspector will place white inspection tags or stickers on some of the containers when he releases the paint to the jobsite.

These inspection release tags may or may not have the same State lot number as shown on the manufacturer's label. The lot number on the release tag should be checked against the Report of Inspection (R-29) which the Inspector should forward to the jobsite within a day or two of releasing the paint.

When the paint is delivered to the jobsite, the Structure Representative is to randomly select one container of each batch and ship it to the Sacramento Transportation Laboratory in its original unopened condition. Sample identification form DCR-TL-101 is to be filled in and sent in with the sample. No paint is to be applied to the structure until the paint in this random sample has been tested by the Transportation Laboratory, and a test report has been issued confirming that the paint complies with the Contract Specifications. The test report is normally mailed to the jobsite; however, results of the testing may be obtained, by phone if necessary. The sample should show his telephone number on the sample identification form if he wishes to be notified of the test results by phone. Results can generally be obtained from the Transportation Laboratory within three to ten days after the Laboratory has received the sample.

One quart of paint is used in the paint testing procedure. The remainder of the paint in the container will be returned to the Contractor for use. The sample identification card should give the address to which the unused paint is to be returned. The unused paint cannot be returned to a P.O. Box Number.

The Standard Specifications permit the use of other than steel containers provided that the containers shall comply with U.S. Department of Transportation or the Interstate Commerce Commission regulations.

BRIDGE CONSTRUCTION BULLETIN **Approved:** _______ FOR **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 155-4** **PAINT** **Date: February 20, 1997** **Expires: March 1, 1998** **Supersedes: AJ96.7**

Subject: Application of Zinc-Rich Primer on Column Casings

It has come to our attention that some contractor's are using rollers to apply zinc-rich primer for repairs and touch up at welds on column casing jobs.

Section 59-2.13, "Application of Zinc-Rich Primer", of the Standard Specifications requires zinc-rich primer to be applied by the spray method and that an agitating pot be used.

Section 59-2.13, of the Standard Specifications, does allow zinc-rich primer to be applied by brush, dauber or roller if the area to be painted is <u>inaccessible</u> to spray application. Column casings are not to be considered inaccessible for spray applications.

On going column casing contracts, any zinc-rich primer not yet top coated and applied by any means other than spray methods, should be removed and reapplied using spray methods in accordance with Section 59-2.13. No additional compensation should be allowed for compliance with Section 59-2.13.

cc: Str. Reps
BCEs
ACMs
Consultant Firms
BGauger, Construction Program Manager

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160-5.0	01/15/05	ELECTRIC WELDING OF PRESTRESS STRAND
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DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

BRIDGE CONSTRUCTION MEMO 160-1.0

PRESTRESSED CONCRETE

January 15, 2005

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BROKEN OR SLIPPED PRESTRESS STRANDS

Structure Construction's policy when prestressing strands break and it has been determined by the Engineer that it is structurally satisfactory to leave the broken strands out, is to give the contractor the option to replace the broken strands or request a contract change order with a rebate to the State.

The rebate should be arrived at as follows:

$$\text{Rebate} = \frac{\text{Total Lineal feet broken strand}}{\text{Total lineal feet strand on the job}} \times \text{Contract item price for prestressing}$$

In the case of slipped strand, when it has been determined that it is not feasible to re-grip and stress to the required force and all the other acceptance criteria are met, the same procedure may be followed.

PATCHING CONCRETE UNDER PRESTRESS BEARING PLATES

Epoxy tends to creep or flow under sustained high stress. Therefore, when patching or replacing concrete immediately behind a prestress bearing plate or in the bearing seat area, the repair shall not be made with material that uses epoxy as a binder. However, epoxy may be used for bonding the repair material to the existing concrete.

Extensive repairs should be re-poured rather than dry-packed. The concrete used in the repair area should have attained the strength required for the structure concrete before the stressing operation is started.

BRIDGE CONSTRUCTION MEMO 160-3.0

PRESTRESSED CONCRETE

January 15, 2005

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PRESSURE CELLS

Structure construction utilizes electro-hydraulic pressure cells combined with a strain indicator (read-out box) to perform QA verification of the contractor's mechanical pressure gage(s) during the post-tensioning operation. Many refer to this equipment incorrectly as a "load cell".

The differences between load cells and pressure cells are discussed below.

A **load cell** converts an applied force into a proportional voltage change. Inside the load cell is a **force** transducer. The force transducer contains internal strain gages. These strain gages consist of fine wire elements that, when stretched, change electrical resistance. Four strain gages are arranged inside the force transducer to form a "Wheatstone Bridge Circuit". This circuit allows for the precise measurement of input and output voltage across the loaded element. The strain gage indicator (read-out box) is used to measure the voltage differential across the transducer circuit. The voltage differential is converted to a direct load reading on the display of the strain gage indicator.

A **pressure cell** uses a **hydraulic pressure transducer** to convert an applied pressure into a proportional voltage change. Strain gages are used inside the pressure transducer in the same way as a force transducer. The strain gage indicator (read out box) is used to measure the voltage differential across the transducer circuit. The voltage differential is converted to a direct load reading on the display of the strain gage indicator.

Structure construction uses pressure cells, not load cells when monitoring post–tensioning operations. However, when METS calibrates the contractor's jacks and gages, they use both a load cell and a pressure cell.

Pressure cell units have been assigned to Senior Bridge Construction Engineers throughout the State. The OSC equipment database, available on the OSC intranet site, can be utilized to locate additional equipment, if needed. If further assistance is required obtaining a pressure cell unit or if repairs are needed, contact the OSC equipment manager at 916-227-7777.

Structure Representatives should make advance arrangements with their Construction Engineer to obtain a pressure cell unit. Senior Bridge Engineers will arrange for their personnel in the area to become proficient in the use of the pressure cell units.

Additional information is given in the current addition of the California Prestress Manual.

STRESSING INCOMPLETE BRIDGES

On rare occasions, usually due to unforeseen emergency situations, contractors may desire to post-tension partially completed bridges. All requests to stress partially completed bridges should be discussed with the Bridge Construction Senior, the Area Construction Manager, and the Project Engineer.

The Office of Structure Design, Memo to Designers 11-18, outlines guidelines and will assist in providing statewide uniformity in responding to requests related to stressing of partially stressed bridges. A copy of Memo to Designers 11-18 can be found at: http://www.dot.ca.gov/hq/esc/techpubs/

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ELECTRIC WELDING OF PRESTRESS STRAND

Section 50-1.05 of the Standard Specifications prohibits electrical welding of prestressing strands after fabrication. Arc welding of the strand is an unsound practice and under normal circumstances shall not be allowed.

The issue of field welding prestressing strand typically arises when contractors want to arc weld the ends of the strands together to a pulling head. Welding the ends together prevents the individual strands from slipping in the bundle when the tendon is pulled through the duct.

The main concern with electrical welding of prestress strands is that stray current from the welding procedure may arc and pit a portion of the strand far from the actual weld location. The pitting damage to the strand may adversely affect the service life and performance of the post-tensioning system.

The use of pulling grips or non-electrical based welding of the strand ends (e.g. oxyacetylene brazing, chemical, etc) are acceptable methods for pulling long presstress strands. Pulling grips are used extensively in the electrical industry and have been successfully used to pull long prestress tendons. Regardless of the method used, all damage to the strands caused by the pulling system must be removed (cut back) from the portion to be incorporated into the final work.

As emphasized above, electrical welding of presstress strands is prohibited by the Standard Specifications and is generally understood to be unsound practice. However, under certain unique situations[1], arc welding a pulling head to the strands to facilitate strand installation may be the alternative with the least detrimental affects. A project specific exception to allow arc welding may be given if approved by the State Bridge Engineer and Deputy Division Chief of Structure Construction.

[1.] An exception was recommended and approved for the new Benicia-Martinez Bridge. This extremely complex structure had significantly long (approximately 1000 feet) and sharp radii tendon paths.

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PRESTRESSED CONCRETE WORKING DRAWINGS

Introduction

Structure Construction's procedure for review and approval of working drawings for prestressed concrete is a coordinated effort between Design and Construction personnel. Primary responsibility for approval of the working drawings rests with the Designer. On externally financed projects that are not State designed, primary responsibility for approval of the working drawings rests with the Local Agency Engineer or the Consultant Designer. The Liaison Engineer will have checked the drawings only in relation to the approval or disapproval of the prestress system and a Technical Specialist in Structures Design will have performed a concurrent, cursory review to ensure it is an approved system.

Working Drawings

Memo to Designers 11-1 "Review of Working Drawings, Prestressed Concrete" covers the procedures required for review and approval of working drawings, including responsibilities of Structure Representatives on construction projects. This memo can be found at the following link:

http://www.dot.ca.gov/hq/esc/techpubs/manual/bridgemanuals/bridge-memo-to-designer/bmd.htm - sec11

Normal procedure is for the Contractor (subcontractor or fabricator) to submit all working drawings directly to the Office of Structure Design, Documents Unit, Mail Station 9-4/4I, 1801 30th Street, Sacramento 95816. The Structure Representative is not to accept submittals.

The Documents Unit Group, Office of Structure Design, administers the working drawing approval procedure for all State jobs, including those that are externally financed. The group maintains a record of all working drawings submitted, and distributes copies to all interested parties, during all phases of the approval procedure. This relieves the Structure Representative of tedious administrative details necessary to insure that working drawings are distributed to the right people at the right time.

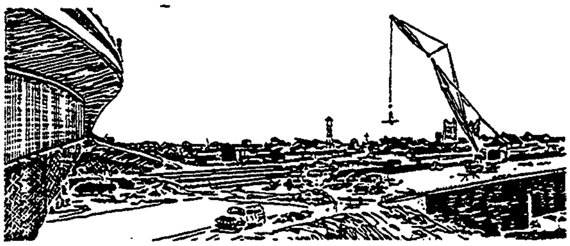
The responsibility for checking working drawings is shared by the Designer and the Structure Representative. Working drawings shall not be returned to the Contractor until the Designer has discussed and resolved the details with the Structure Representative. The comments returned to the Contractor must be acceptable to both the Designer and the Structure Representative.

Instructions to Structure Representatives

1. Comply with applicable instructions in Design Memo 11-1. Communicate directly with the Design Branch Chief, Designer, Liaison Engineer, or Design Consultant when necessary.

2. If the Contractor submits final working drawings to the Resident Engineer or Structure Representative, after reviewing the drawings to be sure that all field changes and minor corrections are noted, transmit all sets promptly to the Office of Structure Design, Documents Unit, Mail Station 9-4/4I, 1801 30th Street, Sacramento 95816.

3. Keep the Resident Engineer informed on the status of the final working drawing submittal so that the contract will not be finalized prior to fulfillment of all contract requirements. Working drawing status can be checked at 'Tracker Web' on the OSC website under the 'Field Resources' tab at the following address: http://onramp.dot.ca.gov/hq/oscnet/.



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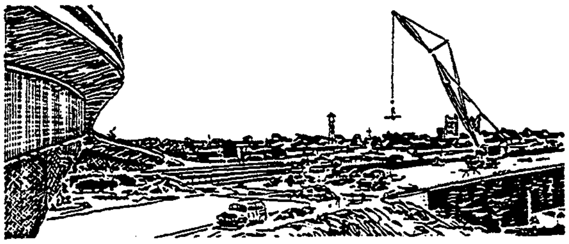
Memo No.	Date	Title
161-1.0	4-27-87	PUMPING PLANT ELECTRICAL AND MECHANICAL EQUIPMENT MATERIALS LISTS AND WORKING DRAWINGS
161-2.0	12-04-81	ELECTRICAL SERVICE FOR PUMPING PLANTS

A. P. Bezzone

A. P. BEZZONE, Chief
Office of Structure Construction

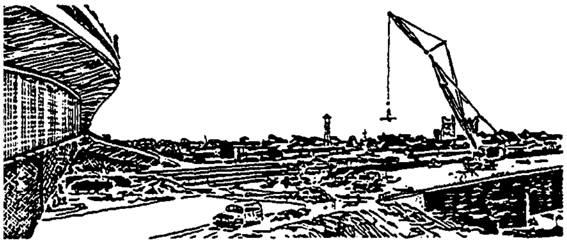


PUMPING PLANT ELECTRICAL AND MECHANICAL EQUIPMENT MATERIALS LISTS AND WORKING DRAWINGS

Materials lists and working drawings for electrical and mechanical equipment, as required by Section 74-1.04 of the Standard Specifications, shall be checked and approved by the Mechanical and Electrical Section.

Normal procedure is for the Contractor (subcontractor or fabricator) to submit all working drawings directly to the Office of Structure Design, Document Unit, P. 0. Box 942874, Sacramento 94274-0001. The Structure Representative is not to accept submittals.

Near the beginning of contracts that have pumping plant electrical and/or mechanical work involved, a representative of the Mechanical and Electrical Section will consult with the Structure Representative and give any necessary instructions at that time.



ELECTRICAL SERVICE FOR PUMPING PLANTS

The contract "Special Provisions" specify the requirements for furnishing electrical service to pumping plants and also specify the provisions for handling the electrical service charges. Generally, the service charges become the obligation of the State upon acceptance of the contract, but may become the State's obligation at some other time during the life of the contract if so specified in the "Special Provisions".

The Structure Representative must notify the District Resident Engineer, so that he may notify the District Office sufficiently in advance to allow time for making arrangements with the utility company for continuing, changing, or discontinuing the service.

Volume II

BRIDGE CONSTRUCTION MEMO 162-0.0

RAILINGS AND BARRIERS

July 1, 1999

Sheet 1 of 1

TABLE OF CONTENTS FOR SECTION No. 162

Memo No.	Date	Title
162-1.0		BLANK
162-2.0	07-01-99	CONCRETE BARRIERS ON STRUCTURES

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

Volume II

CONCRETE BARRIERS ON STRUCTURES

Concrete barriers Type 732 and 736 having constant sloping faces (1:6.25) on the traffic side are approved for use at the edge of deck of structures and retaining walls. Other approved safety shape concrete barriers for use on structures and retaining walls are Types 25, 27 and 28.

The Engineering Service Center has approved the use of the “experimental” Type 60A and Type 60GA for median applications on new structures only.

The type 60A and 60GA are heavier than the Type 50 and existing bridge decks may not carry the additional load. Any proposal to use the type 60A or 60GA on existing bridge decks should be discussed with your designer or design oversight engineer.

OFFICE OF STRUCTURE CONSTRUCTION ***Bridge Construction Records and Procedures Manual***	B98-28
BRIDGE CONSTRUCTION BULLETIN **Approved:_______________________________** **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 162-3** **RAILINGS AND BARRIERS** **Date: October 1, 1998** **Expires: November 1, 1999** **Supersedes: None**

Subject: Finishing of Concrete Barriers

The final surface finish of concrete barriers shall conform to the provisions of section 83-2.02(D), Finishing, of the Standard Specifications. The specification allows the contractor to propose an alternative finishing method.

An allowable alternative is to provide a final surface finish utilizing non-abrasive methods such as a wet sponge finish with cementitious materials. The final surface must be smooth, with an even surface of uniform texture and appearance, free of unsightly bulges and other surface imperfections. The method is subject to approval by the Engineer.

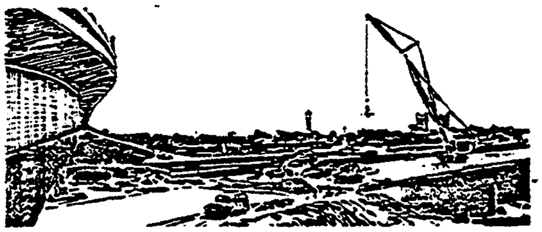
c: BCR&P Manual Holders
Consultant Firms
BFelker, Construction Program Manager

Volume II

TABLE OF CONTENTS FOR SECTION No. 165

Memo No.	Date	Title
165-1.0	2/21/1986	REINFORCING STEEL HOOK DETAILS
165-2.0	2/21/1986	IDENTIFICATION OF REINFORCING STEEL BARS
165-3.0	2/21/1986	REINFORCING STEEL BAR CHART
165-4.0	2/21/1986	WELDED WIRE FABRIC
165-5.0	2/21/1986	FIELD SAMPLING OF REINFORCING STEEL
165-6.0	4/01/1988	THREADED COUPLERS FOR REINFORCING STEEL
165-7.0	1/04/1993	QUALIFICATION OF BAR REINFORCEMENT SPLICES
165-7.2	4/08/1997	MECHANICAL REBAR COUPLERS-SPECIFICATION CLARIFICATIONS
165-7.4	10/15/2002	PREQUALIFICATION LIST FOR STEEL REINFORCING COUPLERS
165-10.0	2/14/2004	REINFORCEMENT SPLICES

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction



REINFORCING STEEL HOOK DETAILS

Reference: ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-83)

The Standard Specifications require that reinforcing steel hooks conform to the provisions of the Building Code Requirements for Reinforced Concrete of the American Concrete Institute.

The attached Chart conforms to the ACI code requirements of Standard hook details for A 615 bars, Grades 40, SO and 60, and for A 706 Grade 60 bars. They are issued for the guidance and information of all field personnel.

A 614 bars, Grade 75 are not to be used in bridge work.

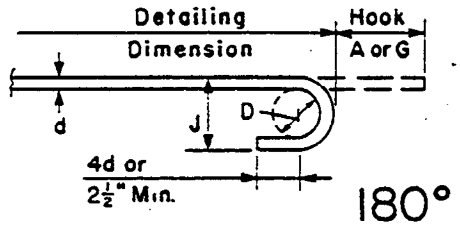
STANDARD HOOKS

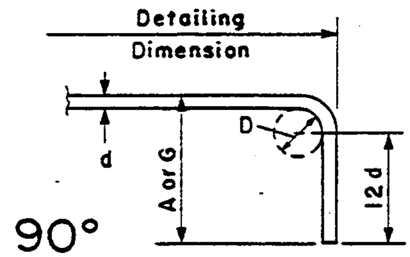
All specific sizes recommended by CRSI below meet minimum requirements of ACI 318-83

RECOMMENDED END HOOKS
All Grades

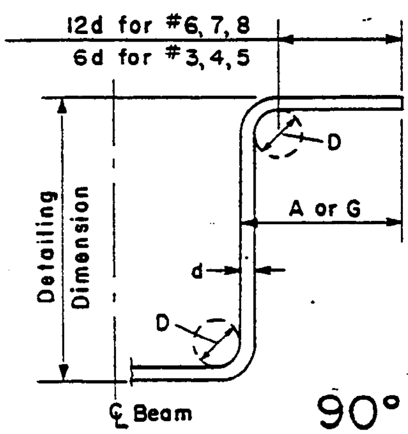
D=Finished bend diameter

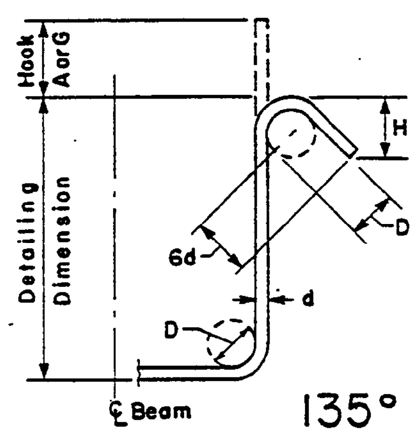
Bar Size	180° HOOKS D	180° HOOKS A or G	180° HOOKS J	90° HOOKS A or G
# 3	2¼	5	3	6
# 4	3	6	4	8
# 5	3¾	7	5	10
# 6	4½	8	6	1-0
# 7	5¼	10	7	1-2
# 8	6	11	8	1-4
# 9	9½	1-3	11¾	1-7
#10	10¾	1-5	1-1¼	1-10
#11	12	1-7	1-2¾	2-0
#14	18¼	2-3	1-9¾	2-7
#18	24	3-0	2-4½	3-5





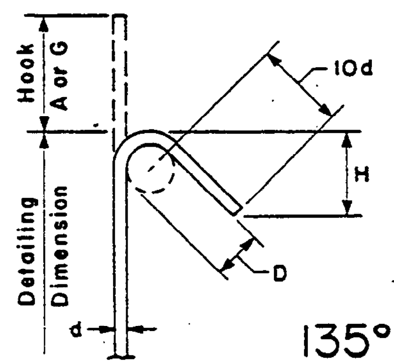
STIRRUP AND TIE HOOKS





STIRRUPS
(TIES SIMILAR)

135° SEISMIC STIRRUP/TIE HOOKS



STIRRUP AND TIE HOOK DIMENSIONS
Grades 40-50-60 ksi

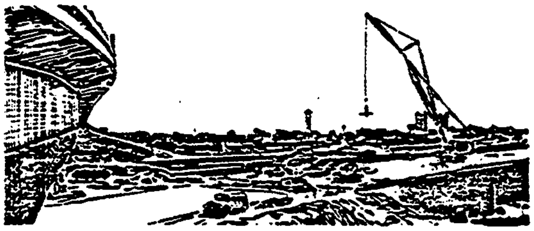
Bar Size	D (in.)	90° Hook: Hook A or G	135° Hook: Hook A or G	135° Hook: H Approx.
#3	1½	4	4	2½
#4	2	4½	4½	3
#5	2½	6	5½	3¾
#6	4½	1-0	7¾	4½
#7	5¼	1-2	9	5¼
#8	6	1-4	10¼	6

135° SEISMIC STIRRUP/TIE HOOK DIMENSIONS
Grades 40-50-60 ksi

Bar Size	D (in.)	135° Hook: Hook A or G	135° Hook: H Approx.
#3	1½	5	3½
#4	2	6½	4½
#5	2½	8	5½
#6	4½	10¾	6½
#7	5¼	1-0½	7¾
#8	6	1-2¼	9

NOTES:

1. 180° hook J dimension (sizes #10, #11, #14 and #18), and A or G dimension (#14 and #18) have been revised to reflect recent test research using ASTM/ACI bend test criteria as a minimum.
2. Tables for Stirrup and Tie Hook dimensions have been expanded to include sizes #6, #7, and #8 to reflect current design practices.



BRIDGE CONSTRUCTION MEMO 165-2.0

REINFORCING STEEL

February 21, 1986

Sheet 1 of 1

Volume II

IDENTIFICATION OF REINFORCING STEEL, BARS

The attached pages of Chapter 1 and Appendix A of the latest Manual of Standard Practice of the Concrete Reinforcing Steel Institute have been reproduced for the information of Structure Construction personnel. (See Attachment No. 1 of this Bridge Construction Memo.)

All current American producers of reinforcing steel bars are listed, and the mills' identifying marks and symbols, for ordinary as well as high strength steel grades, are illustrated.

MATERIAL SPECIFICATIONS FOR REINFORCING BARS (Cont.)

IDENTIFICATION MARK* --ASTM STANDARD REBARS

The ASTM specifications for billet-steel, rail-steel, axle-steel and low-alloy reinforcing bars (A 615, A 616, A 617 and A 706, respectively) require identification marks to be rolled into the surface of one side of the bar to denote the producer's mill designation, bar size, type of steel, and minimum yield designation. Grade 60 bars show these marks in the following order.

1st - Producing Mill (usually a letter)

2nd - Bar Size Number (#3 through #18)

3rd -Type of Steel: **S** for Billet (A 615)

I for Rail (A 616)

IR for Rail meeting Supplementary Requirements SI(A616)

A for Axle (A 617)

W for Low-Alloy (A 706)

4th - Minimum Yield Designation

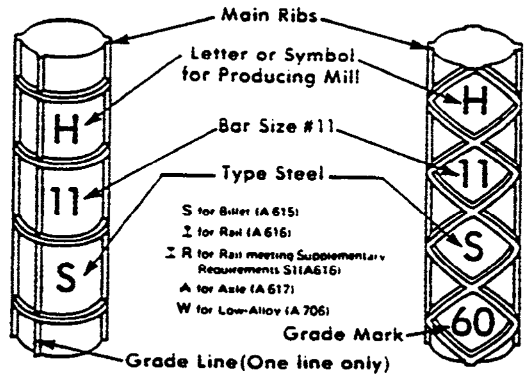
Minimum yield designation is used for Grade 60 and Grade 75 bars only. Grade 60 bars can either have one (1) single longitudinal line (grade line) or the number 60 (grade mark). Grade 75 bars can either have two (2) grade lines if the grade mark 75.

A grade iine is smaller and is located between the two main ribs which are on opposite sides of all bars made in the United States. A grade fine must be continued through at least 5 deformation spaces. and it may be placed on the side of the bar opposite the bar marks. A grade mark is the 4th mark on the bar.

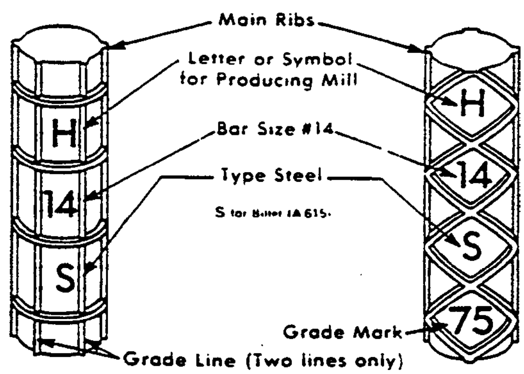
Grade 40 and 50 bars are required to have only the first three identification marks (no minimum yield designation).

VARIATIONS: Bar identification marks may also be oriented to read horizontally (at 9O° to those illustrated).

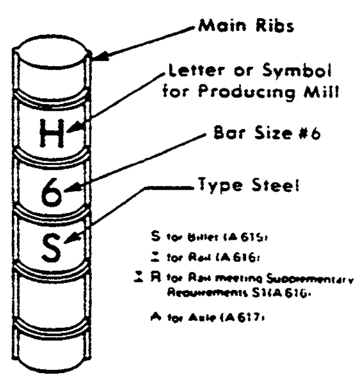
Grade mark numbers may be placed within separate con. secutive deformation spaces to read vertically or horizon. tally.



GRADE 60 AND A 706



GRADE 75



GRADE 40 AND 50

ACI BUILDING CODE - REQUIREMENTS FOR REINFORCING BARS

The current ACI Building Code requires billet-steel reinforcing to conform to the ASTM A 615 specification.

Rail-steel reinforcing bars must meet A 616 including supplementary requirement (Sl). As shown in the mechanical requirements table on page 1-2, the supplementary requirement (S1) prescribes more-restrictive bend tests. SI also requires that A 616 reinforcing bars furnished to these supptementary requirements must be designated for type of steel by the symbol "R", in addition to the rail symbol.

The ACI Code does not have special requirements for axle-steel (A 617) and low-alloy (A 706) reinforcing bars, nor take any exceptions to the ASTM specifications for these bars.

'See Appendix A for complete identification marks of concrete reinforcing bars produced by all U.S. manufacturers. The marks, listed alphabetically by producing mill include the identification requirements of ASTM and the deformation pattern used by each mill.

U.S. MANUFACTURERS OF CONCRETE REINFORCING BARS

IDENTIFICATION OF U.S. REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings. See page 1-3.

1 A.B. STEEL MILL, INC.

A 

#3 and #4 bars only
Grade mark line used for #3

1 A.B. STEEL MILL, INC.

A 

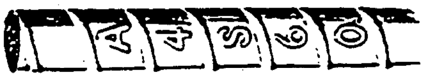
Bars #5 through #10 only

2 ARMCO INC.
(Midwestern Steel Division)

S 

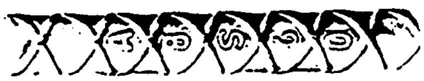
#3 and #4 bars only
Grade mark line on opposite side

3 ATLANTIC STEEL COMPANY

S 

Coiled bars (#3 through #5 only)

3 ATLANTIC STEEL COMPANY

S 

Straight bars (#3 through #11 only)

3 ATLANTIC STEEL COMPANY

MILL	BAR SIZES
Cartersville	#3 through #7 only
Atlanta	#8 through #11 only

4 AUBURN STEEL COMPANY, INC.

S 

Bars #3 through #5 only

4 AUBURN STEEL COMPANY, INC.

S 

Bars #6 through #11 only

5 BAYOU STEEL CORPORATION

S 

Bars #4 through #6 only
Grade mark line on opposite side

6 BIRMINGHAM STEEL CORPORATION
(Barbary Coast Steel Corporation)

S 

6 BIRMINGHAM STEEL CORPORATION
(Illinois Division)

S 

Bars #4 through #11 only

6 BIRMINGHAM STEEL CORPORATION
(Mississippi Steel Division)

S 

6 BIRMINGHAM STEEL CORPORATION
(Norfolk Steel Corporation)

S 

Bars #4 through #11 only

6 BIRMINGHAM STEEL CORPORATION
(Salmon Bay Steel Division)

S 

Bars #3 through #9 only

6 BIRMINGHAM STEEL CORPORATION
(Salmon Bay Steel Division)

S 

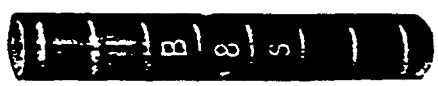
Bars #10 through #18 only

6 BIRMINGHAM STEEL CORPORATION
(Southern United Steel Division)

S 

Bars #5 through #11 only

7 BORDER STEEL MILLS, INC.

S 

U.S. MANUFACTURERS OF CONCRETE REINFORCING BARS

IDENTIFICATION OF U.S. REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings. See page 1-3.

8 CALUMET STEEL COMPANY

S 

Bars #4 through #10 only

9 CASCADE STEEL ROLLING MILLS, INC.

S 

10 CF&I STEEL CORPORATION

S 

Bars #3 through #7 only

10 CF&I STEEL CORPORATION

S 

#8 bar only

10 CF&I STEEL CORPORATION

S 

Bars #9 through #11 only

11 CHAPARRAL STEEL COMPANY

S 

Grade mark line on opposite side

12 CHICAGO HEIGHTS STEEL

I 

Bars #4 through #8 only

13 COMMERCIAL STEEL CORPORATION

I 

Bars #3 through #6 only

14 CONNECTICUT STEEL CORPORATION

S 

Bars #3 through #11 only
Grade mark line on opposite side

15 FLORIDA STEEL CORPORATION

(Charlotte Steel Mill Division)

S 

Bars #4 through #11 only

15 FLORIDA STEEL CORPORATION

(Jacksonville Steel Mill Division)

S 

Bars #3 through #11 only

15 FLORIDA STEEL CORPORATION

(Knoxville Steel Mill Division)

S 

Bars #3 through #11 only

15 FLORIDA STEEL CORPORATION

(Tampa Steel Mill Division)

S 

Bars #4 through #11 only

15 FLORIDA STEEL CORPORATION

(West Tennessee Steel Mill Division)

S 

Bars #4 through #18 only

16 FRANKLIN STEEL COMPANY

I 

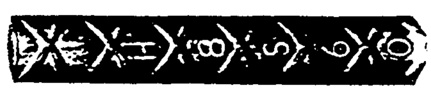
Bars #4 through #11 only

17 GEORGETOWN STEEL CORPORATION

N 

Bars #3 through #5 only

18 HAWAIIAN WESTERN STEEL, LTD.

S 

U.S. MANUFACTURERS OF CONCRETE REINFORCING BARS

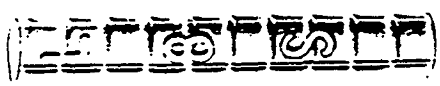
IDENTIFICATION OF U.S. REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings. See page 1-3.

19 INLAND STEEL COMPANY

S 

20 LACLEDE STEEL COMPANY

S 

Straight bars

20 LACLEDE STEEL COMPANY

S 

Bars #3 and #4 only, coiled bars

21 LTV STEEL COMPANY

S 

Bars #5 through #11 only

22 MARION STEEL COMPANY

S 

Bars #4 through #11 only

23 NEW JERSEY STEEL CORPORATION

S 

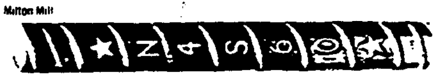
Bars #3 through #11 only

24 NORTH STAR STEEL COMPANY
(Beaumont Mill)

S 

Bars #3 and #4 only

24 NORTH STAR STEEL COMPANY
Milton Mill

S 

Bars #3 through #9 only

24 NORTH STAR STEEL COMPANY
Milton Mill

S 

Bars #10 through #18 only

24 NORTH STAR STEEL COMPANY
(Monroe Mill)

S 

Bars #4 through #18 only
Grade mark line on opposite side

24 NORTH STAR STEEL COMPANY
(St. Paul Mill)

S 

Bars #4 through #11 only
Grade mark line on opposite side

24 NORTH STAR STEEL COMPANY
(St Paul Mill)

S 

#14 and #18 bars only
Grade mark line on opposite side

24 NORTH STAR STEEL COMPANY
(St Paul Mill)

N 

Bars #6 through #18 (Patented)—Longitudinal groove on one side only
Marking system not in conformance with ASTM Specifications

24 NORTH STAR STEEL COMPANY
(Wilton Mill)

S 

Mill symbol "I" either appears as first mark (shown) or as last mark (under S)

25 NORTHWESTERN STEEL & WIRE CO.

S 

Bars #3 through #10 only

26 NUCOR CORPORATION
(Nebraska Mill)

S 

Bars #4 through #11 only

26 NUCOR CORPORATION
(Utah Mill)

S 

Bars #4 through #18 only

U.S. MANUFACTURERS OF CONCRETE REINFORCING BARS

IDENTIFICATION OF U.S. REINFORCING BARS

ASTM and AASHTO Specifications require that all reinforcing bars be identified by permanent, mill imprinted markings. See page 1-3.

27 OWEN ELECTRIC STEEL CO. OF S.C.

S 

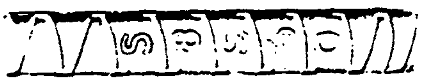
Bars #3 through #14 only

28 ROANOKE ELECTRIC STEEL CORP.

S 

Bars #3 through #11 only

29 SEATTLE STEEL, INC.

S 

Bars #3 through #11 only

29 SEATTLE STEEL, INC.

S 

#14 and #18 bars only

30 SHEFFIELD STEEL CORPORATION

S 

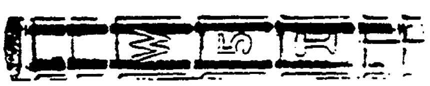
Bars #3 through #14 only

31 SILVER, INC., W.

I 

Bars #3 through #6 only

31 SILVER, INC., W.

I 

Bars #3 through #6 only

31 SILVER, INC., W.

I 

Bars #3 through #6 only
Grade mark line on opposite side

32 SMI STEEL - ARKANSAS

I 

Bars #3 through #11 only

32 SMI STEEL - ARKANSAS

I 

Bars #3 through #6 only

33 STRUCTURAL METALS, INC.

S 

Bars #3 through #11 only

33 STRUCTURAL METALS, INC.

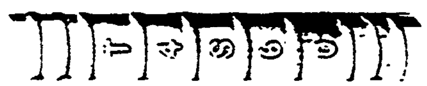
S 

#14 and #18 bars only

34 TAMCO

S 

34 TAMCO

S 

Bars #4 and #5 only

34 TAMCO

S 

Bars #6 through #18 only

35 THOMAS STEEL CORPORATION

S 

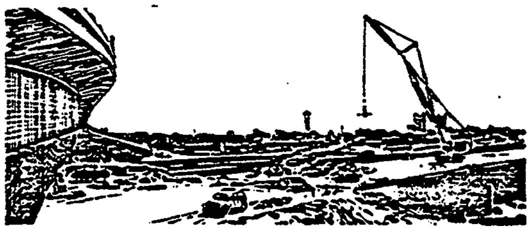
36 USX CORPORATION

S 

U.S. MANUFACTURERS OF CONCRETE REINFORCING BARS

NUMBERS REFER TO BAR MARK PHOTOS

1. A.B. STEEL MILL, INC.
   Cincinnati, Ohio
2. ARMCO. INC.
   Kansas City, Missouri
3. ATLANTIC STEEL COMPANY
   Atlanta, Georgia
4. AUBURN STEEL COMPANY
   Auburn, New York
5. BAYOU STEEL CORPORATION
   La Place. Louisiana
6. BIRMINGHAM STEEL CORPORATION
   Birmingham, Alabama
7. BORDER STEEL MILLS. INC.
   El Paso, Texas
8. CALUMET STEEL COMPANY
   Chicago Heights. Illinois
9. CASCADE STEEL ROLLING MILLS INC.
   McMinnville, Oregon
10. CF & I STEEL CORPORATION
    Pueblo, Colorado
11. CHAPARRAL STEEL COMPANY
    Midlothian, Texas
12. CHICAGO HEIGHTS STEEL
    Chicago Heights, Illinois
13. COMMERCIAL STEEL CORPORATION
    Glassport, Pennsylvania
14. CONNECTICUT STEEL CORPORATION
    Wallingford, Connecticut
15. FLORIDA STEEL CORPORATION
    Tampa. Florida
16. FRANKLIN STEEL COMPANY
    Franklin. Pennsylvania
17. GEORGETOWN STEEL CORPORATION
    Georgetown. South Carolina
18. HAWAIIAN WESTERN STEEL. LTD.
    Ewa Beach, Hawaii
19. INLAND STEEL COMPANY
    Chicago, Illinois
20. LACLEDE STEEL COMPANY
    St. Louis, Missouri
21. LTV STEEL COMPANY
    Cleveland, Ohio
22. MARION STEEL COMPANY
    Marion, Ohio
23. NEW JERSEY STEEL CORPORATION
    Sayreville, New Jersey
24. NORTH STAR STEEL COMPANY
    Minneapolis, Minnesota
25. NORTHWESTERN STEEL&WIRE CO.
    Sterling, Illinois
26. NUCOR STEEL CORPORATION
    Norfolk, Nebraska
27. OWEN ELECTRIC STEEL COMPANY OF S.C.
    Cayce, South Carolina
28. ROANOKE ELECTRIC STEEL CORP.
    Roanoke, Virginia
29. SEATTLE STEEL, INC.
    Seattle, Washington
30. SHEFFIELD STEEL CORPORATION
    Sand Springs. Oklahoma
31. SILVER, INC., W.
    El Paso, Texas
32. SMI STEEL-ARKANSAS
    Magnolia. Arkansas
33. STRUCTURAL METALS. INC.
    Seguin, Texas
34. TAMCO
    Etiwanda, Cailfornia
35. THOMAS STEEL CORPORATION
    Lemont,. Illinois
36. USX CORPORATION
    Pittsburgh, Pennsylvania



REINFORCING STEEL BAR CHART

The attached chart (Attachment #1) of reinforcing steel properties has been a valuable tool of Structure Construction personnel for many years.

Field personnel desiring additional copies of this chart may requisition them from the Office of Structure Construction.

REINFORCING STEEL

STANDARD A305 REINFORCING BARS

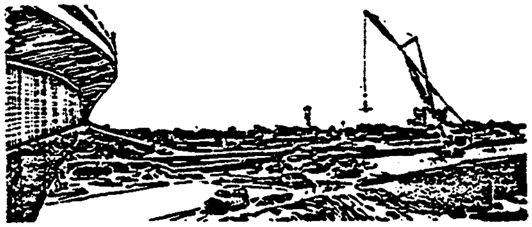
BAR SIZES		WEIGHT	NOMINAL DIMENSIONS — ROUND SECTIONS		
OLD (INCHES)	NEW (NUMBERS)	POUNDS PER FOOT	DIAMETER INCHES	CROSS SECTIONAL AREA - SQ. INCHES	PERIMETER INCHES
1/4	2	.167	.250	.05	.786
3/8	3	.376	.375	.11	1.178
1/2	4	.668	.500	.20	1.571
5/8	5	1.043	.625	.31	1.963
3/4	6	1.502	.750	.44	2.356
7/8	7	2.044	.875	.60	2.749
1	8	2.670	1.000	.79	3.142
1	9	3.400	1.128	1.00	3.544
1 1/8	10	4.303	1.270	1.27	3.990
1 1/4	11	5.313	1.410	1.56	4.430
1 1/2	14	7.650	1.692	2.25	5.316
2	18	13.600	2.256	4.00	7.088

Bar Size	Out-to-Out Dimension
#3, #4, #5	Add 12% to the nominal bar diameter
#6 through #8	Add 10% to the nominal bar diameter

The new bar numbers are based on the number of 1/8 inches included in the nominal diameter of the bar.

Bar number 2 in plain rounds only. Bars numbered 9, 10 and 11 are round bars and equivalent in weight and nominal cross-sectional area to the old type 1", 1-1/8" and 1-1/4" square bars.

These weights have been approved through the U.S. Department of Commerce Simplified Practice Recommendation R 26-50.



BRIDGE CONSTRUCTION MEMO 165-4.0

REINFORCING STEEL

February 21, 1986

Volume II

WELDED WIRE FABRIC

The attached pages of Chapter 2 (Welded Wire Fabric) of the latest Manual of Standard Practice of the Concrete Reinforcing Steel Institute have been reproduced for the information of Structure Construction personnel. (See Attachment No. 1 of this Bridge Construction Memo.)

CHAPTER 2

WELDED WIRE FABRIC

This Chapter contains information on ASTM specifications and style identification plus design, detailing and estimating data for welded wire fabric used in building construction. The use of wire gauge sizes in welded wire fabric has been replaced by a new W-number system of designating wire sizes. Smooth welded wire fabric wire sizes are denoted by the letter W- followed by a number indicating the cross-sectional area of the wire in hundredths of a square inch. Deformed wire sizes are similarly denoted by the letter D- followed by a number indicating the wire's cross-sectional area in hundredths of a square inch.

1. ASTM SPECIFICATIONS

Welded wire fabric used for concrete reinforcement consists of cold-drawn wire in orthogonal patterns, square or rectangular, resistance welded at all intersections. Welded wire fabric (WWF) is commonly but erroneously called "mesh" which is a much broader term not limited to concrete reinforcement. Welded wire fabric must conform to ASTM A185 if made of smooth wire or A497 if made of deformed wire. These Specifications require shear tests on the welds essential to proper anchorage for bond in concrete. ASTM yield strength is 65,000 psi for smooth fabric (A185) and is 70,000 psi for deformed fabric (A497).

Unless otherwise specified, welded wire fabric conforming to ASTM A185 will be furnished.

2. DESIGNING WITH WELDED WIRE FABRIC

1. The engineers' selection of structural welded wire. fabric styles should include production considerations as well as steel area requirements. Maximum economies in production and job-handling can be achieved by utilizing repetition of styles and duplication of sheet and/or roll dimensions to the fullest extent possible.
2. WWF is manufactured in sheets and rolls. Wire sizes smaller than W1.4 are usually manufactured only in rolls and wire sizes larger than W4 are usually manufactured in sheets.
3. Maximum size sheet (width and/or length) may be limited by shipping restrictions as well as manufacturing limitations.
4. Design drawings should show edge and end anchorage requirements for extension into end or side supports.

3. STYLE IDENTIFICATION

Smooth wire is denoted by the letter "W" followed by a number indicating cross-sectional' area in hundredths of a "square inch. Deformed wire is similarly denoted by the letter "D" followed by a number indicating cross-sectional area in hundredths of a square inch.

Welded wire fabric is usually denoted on design drawings as follows: WWF followed by spacings of longitudinal wires and then transverse wires and last by the sizes of longitudinal and transverse wires.

An example style designation (see Figure 1) is: WWF 6x12-W16xW8. This designation identifies a style of fabric in which:

Spacing of longitudinal wires . . . . = 6"
Spacing of transverse wires . . . . = 12"
Longitudinal wire size . . . . = W16
Transverse wire size . . . . = W8

A deformed-fabric style would be designated in the same manner with the appropriate D-number wire sizes.

It is very important to note that the terms "longitudinal" and "transverse" are related to the method of fabric manufacture and have no reference to the position of the wires in a completed concrete structure.

In the manufacture of fabric, longitudinal wires move continuously through an automatic welding machine. Transverse wires are individually welded perpendicularly across all longitudinal wires each time the longitudinal wires move through the welder a distance-equal to one transverse wire spacing.

4. DETAILING WELDED WIRE FABRIC

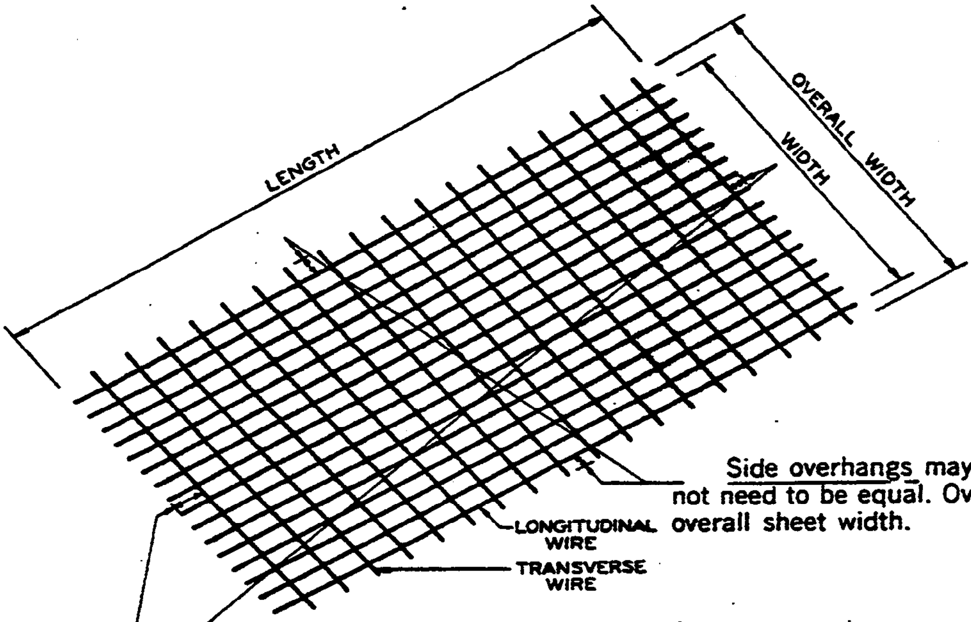
The quantity of fabric detailed and supplied shall include the net area shown on the drawings plus sufficient material to include adequate splices.

WIDTH:

1. Width is defined as the center-to-center distance between the outside longitudinal wires.
2. Overall width is defined as the width plus side overhangs.
3. The side overhangs of transverse wires Will be no greater than one inch unless otherwise specified. Transverse wires may be specified to have a specific overhang. Also, the overhang. may vary from one side to the other if necessary.

LENGTH:

1. Welded wire fabric rolls can be manufactured in any lengths, up to the maximum weight per roll, convenient for handling. The lengths of rolls vary with the individual manufacturing practices of different producers. Typical lengths may be 100, 150 and 200 feet. Sheet or roll length is defined as the length, tip to tip, of longitudinal wires. This length should be an even multiple of the transverse wire spacings.
2. The sum of the two end overhangs on either sheets or rolls should be equal to one transverse wire spacing. Unless otherwise specified, each end overhang equals one-half of a transverse spacing.



Side overhangs may be varied as required and do not need to be equal. Overhang lengths limited only by overall sheet width.

End overhangs — The sum of the end overhangs should equal one transverse wire space. Unless otherwise specified, each end overhang equals one-half of a transverse space.

Industry, Method of Designating Style:
Example -- WWF 6x12-W16xW8

Longitudinal wire spacing . . . . 6"	**Longitudinal wire size W16**
Transverse wire spacing . . . .12"	**Transverse wire size W8**

Figure 1

5. COMMON STOCK STYLES OF WELDED WIRE FABRIC

Certain styles of welded wire fabric as shown in Table I have been recommended by the Wire Reinforcement institute as common stock styles. Use of these styles is normally based on empirical practice and quick availability rather than on specific steel area designs. Styles of fabric produced to meet other specific steel area requirements are ordered for designated projects, or, in some localities, may be available from inventory. For comparative purposes, the previously used style designations by steel wire gage are included in Table I.

TABLE I - COMMON STOCK STYLES OF WELDED WIRE FABRIC

STYLE DESIGNATION		Steel Area Sq. in. Per Ft.		weight Approx. Lbs. Per 100 Sq. Ft.
New Designation (By W-numbed	Old Designation (By Steel Wire Gage)	Longit.	Transv.	
ROLLS				
6x6-W 1.4xW1.4	6x6-10x10	.03	.03	21
6x60W2xW2	6x6-8x8"	.04	.04	29
6x6-W2.9xW2.9	6 x 6 - 6 x 6	.06	.06	42
6x60W4xW4	6x6-4x4	.08	.08	58
4x4-w.1.4xw1.4.	4x4-10x 10	.04	.04	31
4x40W2xW2	4x4-8x8'	.06	.06	43
4x4.w2.9xw2.9	4x4-6x6	.09	.09	62
4x4-w4xw4	4x4-4x4	.12	.12	86
SHEETS				
6x6-W2.9xW2.9	6x6-6x6	.06	.06	42
6x60W4xW4	6x6-4x4	.08	.08	58
6x60W5.5xW5.5	6x6-2x2**	.11	.11	80
4x4-W4xW4	4x4-4x4	.12	.12	86

*Exact W-number size for 8-gage is W2.1.
**Exact W-number size for 2-gage is W5.4.

TABLE II-SCHEDULED UNIT WEIGHTS FOR ESTIMATING WELDED WIRE FABRIC*

(Approximate weights in pounds per 100 square feet)

Wire Size Number		Nominal Diameter, Inches	Spacing and Weight of Longitudinal Wires					Spacing and Weight of Transverse Wires					
Smooth	Deformed		2"	3"	4"	6"	12"	3"	4"	6"	8"	10"	12"
W20	D20	0.505	422	286	218	150	82	281	211	141	105	84	70
W18	D18	0.479	379	257	196	135	73	253	190	126	95	76	63
W16	D16	0.451	337	228	174	120	65	225	169	112	84	67	56
W14	D14	0.422	295	200	152	105	57	197	148	98	74	59	49
W12	D12	0.391	253	171	131	90	49	169	126	84	63	51	42
W11	D11	0.374	232	157	120	82	45	155	116	77	58	46	39
W10.5		0.366	221	150	114	79	43	148	111	74	55	44	37
W10	D10	0.357	211	143	109	75	41	141	105	70	53	42	35
W9.5		0.348	200	136	103	71	39	134	100	67	50	40	33
W9	D9	0.338	190	129	98	67	37	126	95	63	47	38	32
W8.5		0.329	179	121	92	64	35	119	90	60	45	36	30
W8	D8	0.319	169	114	87	60	33	112	84	56	42	34	28
W7.5		0.309	158	107	82	56	31	105	79	53	40	32	26
W7	D7	0.299	148	100	76	52	29	98	74	49	37	30	25
W6.5		0.288	137	93	71	49	27	91	69	46	34	27	23
W6	D6	0.276	126	86	65	45	24	84	63	42	32	25	21
W5.5		0.265	116	79	60	41	22	77	58	39	29	23	19
W5	D5	0.252	105	71	54	37	20	70	53	35	26	21	18
W4.5		0.239	95	64	49	34	18	63	47	32	24	19	16
W4	D4	0.226	84	57	44	30	16	56	42	28	21	17	14
W3.5		0.211	74	50	38	26	14	49	37	25	18	15	12
W3		0.195	63	43	33	22	12	42	32	21	16	13	11
W2.9		0.192	61	41	32	22	12	41	30	20	15	12	10
W2.5		0.178	53	36	27	19	10	35	26	18	13	11	9
W2.1		0.162	43	29	22	15	8	29	22	15	11	9	7
W2		0.160	42	29	22	15	8	28	21	14	11	8	7
W1.5		0.138	32	21	16	11	6	21	16	11	8	6	5
W1.4		0.134	30	21	16	11	6	20	15	10	7	6	5

*Based on 60” width, 1” ‘side overhang each side (62” overall width), and standard end overhangs.

Note: This table is to be used for estimating purposes only. Exact weights of welded wire fabric will vary from those shown above, depending upon width of rolls or sheets and lengths of overhangs. No allowance is made in this table for the extra weight of fabric required for laps or splices.

EXAMPLE: Approximate weight of 6x6 - W4xW4
Longitudinal = 30
Transverse = 28
58 lbs. per 100 sq. ft.

BRIDGE CONSTRUCTION MEMO 165-4.0
ATTACHMENT NO. 1 (Rev. 2-21-86)
Sheet 4 of 5

CHAPTER 2 - WELDED WIRE FABRIC (Cont.)

TABLE III - SECTIONAL AREA AND WEIGHT OF WELDED WIRE FABRIC

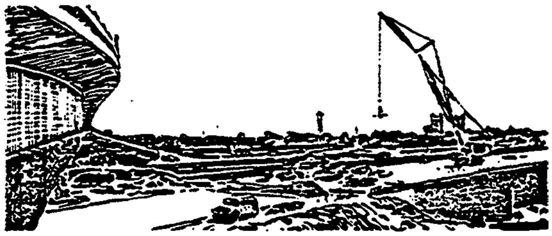
Wire Size Number[a]		Nominal Diameter Inches	Nominal Weight Lbs./Lin. Ft.	Area In Sq. In. Per Ft. Of Width For Various Spacings — Center-To-Center Spacing						
Smooth	Deformed			2"	3"	4"	6"	8"	10"	12"
W20	D20	0.505	.680	1.20	.80	.60	.40	.30	.24	.20
W18	D18	0.479	.612	1.08	.72	.54	.36	.27	.216	.18
W16	D16	0.451	.544	.96	.64	.48	.32	.24	.192	.16
W14	D14	0.422	.476	.84	.56	.42	.28	.21	.168	.14
W12	D12	0.391	.408	.72	.48	.36	.24	.18	.144	.12
W11	D11	0.374	.374	.66	.44	.33	.22	.165	.132	.11
W10.5		0.366	.357	.63	.42	.315	.21	.157	.126	.105
W10	D10	0.357	.340	.60	.40	.30	.20	.15	.12	.10
W9.5		0.348	.323	.57	.38	.285	.19	.142	.114	.095
W9	D9	0.338	.306	.54	.36	.27	.18	.135	.108	.09
W8.5		0.329	.289	.51	.34	.255	.17	.127	.102	.085
W8	D8	0.319	.272	.48	.32	.24	.16	.12	.096	.08
W7.5		0.309	.255	.45	.30	.225	.15	.112	.09	.075
W7	D7	0.299	.238	.42	.28	.21	.14	.105	.084	.07
W6.5		0.288	.221	.39	.26	.195	.13	.097	.078	.065
W6	D6	0.276	.204	.36	.24	.18	.12	.09	.072	.06
W5.5		0.265	.187	.33	.22	.165	.11	.082	.066	.055
W5	D5	0.252	.170	.30	.20	.15	.10	.075	.06	.05
W4.5		0.239	.153	.27	.18	.135	.09	.067	.054	.045
W4	D4	0.226	.136	.24	.16	.12	.08	.06	.048	.04
W3.5		0.211	.119	.21	.14	.105	.07	.052	.042	.035
W3		0.195	.102	.18	.12	.09	.06	.045	.036	.03
W2.9		0.192	.099	.174	.116	.087	.058	.043	.035	.029
W2.5		0.178	.085	.15	.10	.075	.05	.037	.03	.025
W2.1		0.162	.070	.126	.084	.063	.042	.031	.025	.021
W2		0.160	.068	.12	.08	.06	.04	.03	.024	.02
W1.5		0.138	.051	.09	.06	.045	.03	.022	.018	.015
W1.4		0.134	.048	.084	.056	.042	.028	.021	.017	.014

Note: The above listing of smooth and deformed wire sizes represents wires normally selected to manufactrue welded wire fabric styles to specific areas of reinforcement. Wire sizes other than those listed above, including larger sizes, may be available if the quantity required is sufficient to justify manufacture.

[a]The number following the prefix W or the prefix D identifies the cross-section area of the wire in hundredths of a square inch.

The nominal diameter of a deformed wire is equivalent to the diameter of a smooth wire having the same weight per foot as the deformed wire.

The ACI Building Code requirements for tension development lengths and tension lap splices of welded wire fabric are not included in this Chapter, These design requirements are covered in Reinforcement Anchorages and Splices available from CRSI. For additional information, see *Welded Wire Fabric Manual of Standard Practice and Structual Welded Wire Fabric Detailing Manual Part I,* both published by the Wire Reinforcement Institute.



BRIDGE CONSTRUCTION MEMO 165-5.0

REINFORCING STEEL

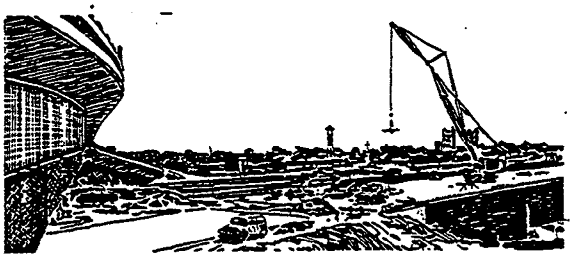
February 21, 1986

Sheet 1 of 1

Volume II

FIELD SAMPLING OF REINFORCING STEEL

Field-samples of No. 14 and No. 18 reinforcing steel must be 36 inches long. This is necessary because the testing laboratory has a testing machine which will not accept NO. 14 and No. 18 bars that are shorter than 36 inches.



THREADED COUPLERS FOR REINFORCING STEEL

Two types of threaded reinforcing steel bar couplers are currently being used. TransLab qualifies them by testing specimens meeting thread criteria of the supplier. It is important to note that this criteria is radically different in the case of the Fox Howlett and Lenton couplers. Gauges should be furnished to the Structure Representative to check thread pitch, shape and engagement length.

FOX HOWLETT COUPLER

Where Fox-Hewlett commercial type" threaded couplers #18C, #14C and #11C are used to splice reinforcing steel, the basis for acceptance is that there be a minimum number of full depth threads over a minimum distance on each element line on the conical surface of the bar end.

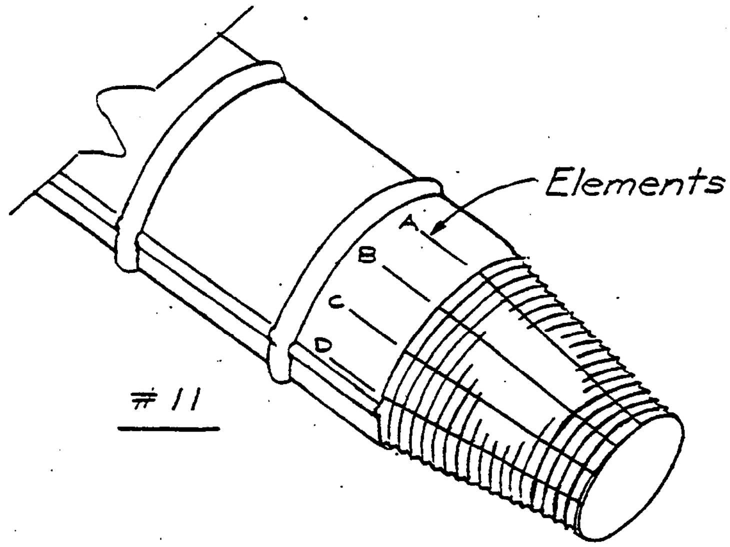
Bar Size	#18	#14	#11
Minimum length of full depth thread	1 1/8"	5/8"	1/2"
Minimum number of full depth threads	14	8	7

The above is based on the following requirements;

1. All element lines on the surface meet the minimum length and number of full depth threads.

2. The total length of thread along an element line may be discontinuous (ie. qualifying threads do not have to be next to each other). Shallow threads do not qualify.

3. Qualifying threads shall be full depth threads that are engaged within the thread gage or coupler after a hand tight condition between 3 1/4 to 4 1/2 turns has been reached.



BAR CONE SHOWING AREAS OF FULL DEPTH THREADS

The bar should be rolled visually looking for the minimum number of, full depth threads along each element line (only four are drawn here).
In this example;

Element A has 13 full depth threads

Element B has 8 full depth threads

Element C has 12 full depth threads

Element D has 17 full depth threads

Proceed around the bar and determine the least number of threads on any element and get the total length on that element (no. of threads times pitch) for compliance. Note that the minimum number of threads are 7 measured along an element between A and B.

ERICO-LENTON COUPLER

FIELD INSPECTION CRITERIA

Externally threaded bar ends must be inspected for:

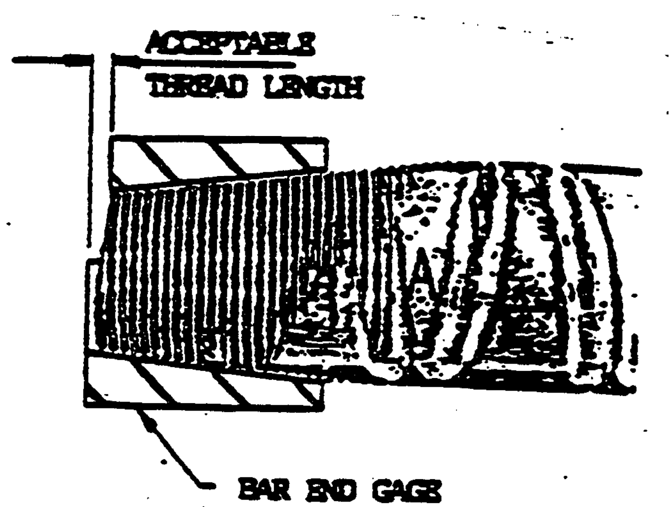
1) Proper engagement length
2) Thread profile
3) Non-concentric threads

INSPECTION CRITERIA

1. THREAD-ENGAGEMENT LENGTH

To insure proper thread length, a gage referred to as a "Bar End Gage" must be used. The gage is inserted over the end of the bar after threading. The bar end must fall within the step on the gage, as shown on the drawing below.

For bars cut out of square, the longest side will be used to determine thread length.

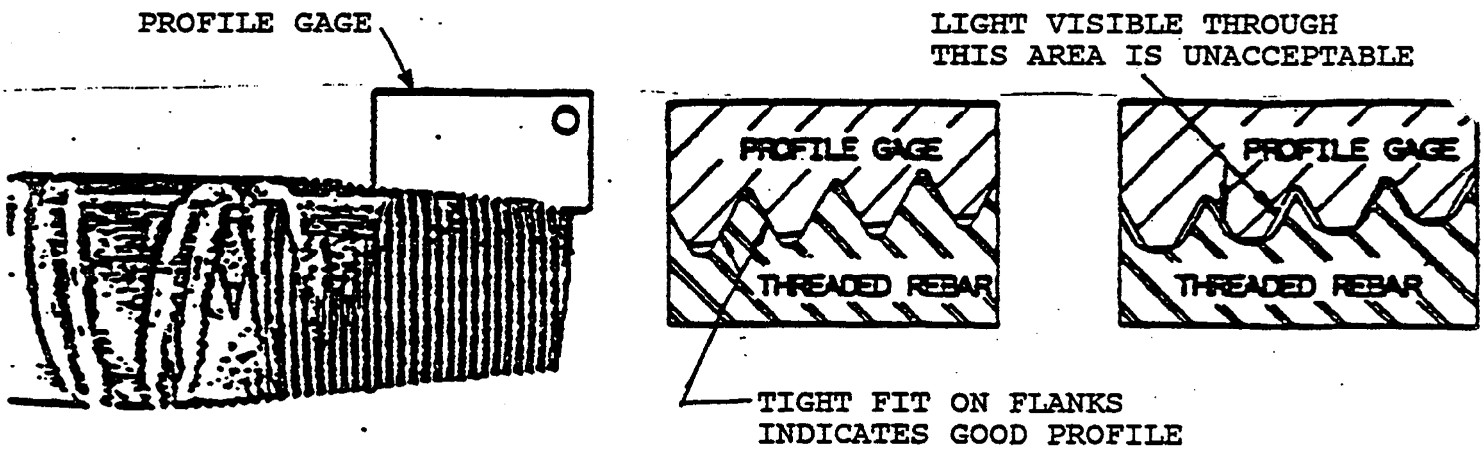


2. THREAD PROFILE

The thread-profile gage is used to determine:

1) Excessively worn chasers.
2) Chasers located in the wrong position in the bar threader.
3) "Washed-out" threads.

NOTE: "Washed-out" threads refers to threads with a wide valley or root, and a sharp crest.



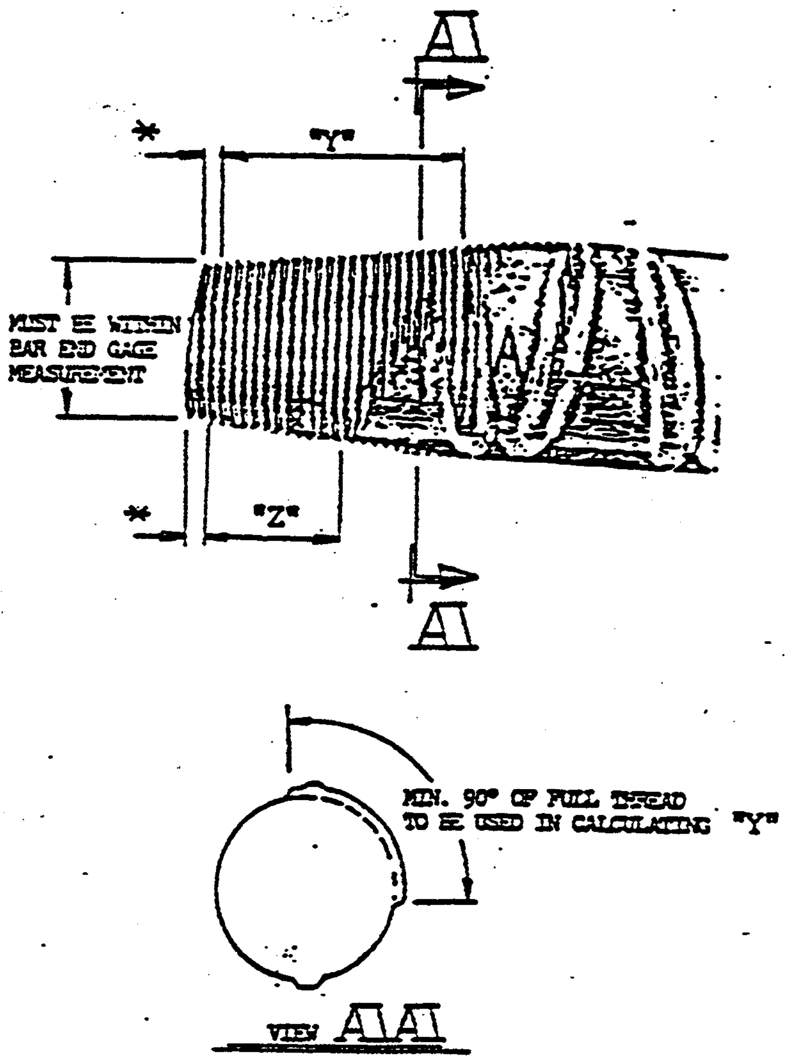
3. NON-CONCENTRIC THREADS

Threaded ends having less threads on one side versus the other will be acceptable if the following requirements are met.

1) By measuring, determine the maximum and minimum length of thread on a bar end. See drawings on the following page.

Note: a) "Washed-out" or stripped-out threads cannot be used in the calculations.

b) Only threads with at least 90 degrees of full circumferential threads, must be used in the calculations.



NOTE: *DISREGARD ANY DAMAGED THREADS IN THIS AREA.
MAXIMUM TWO THREADS.

2) The minimum length of thread must be equal to or greater than "Z". See chart below.

3) The maximum thread length ("Y") added to the minimum length ("Z") must be equal to or greater than "Y+Z". See chart below.

BAR NO.	"Z" (inches)	"Y+Z" (inches)
#3	3/16	9/16
#4	l/4	11/16
#5	5/16	15/16
#6	7/16	1 -3/16
#7	7/16	1 -5/16
#8	l/2	1 -l/2
#9	9/16	l-11/16
#10	11/16	l-15/16
#ll	3/4	2 -l/8
#14	l-1/16	2 -9/16
#18	l-l/2	3 -7/16

Volume II

QUALIFICATION OF BAR REINFORCEMENT SPLICES

The 1992 Standard Specifications allow splicing of reinforcing bars by lapping, butt welding, mechanical butt and mechanical lap splicing, at the option of the Contractor, and as permitted by the Special Provisions.

Section 52-1.08 covers all of the above methods of splicing reinforcing bars; however, requirements for each method are somewhat intermingled within this specification. Careful study by the Structure Representative is needed to sort out specifics for his/her project requirements.

As an aid and quick reference, attachments No. 1 and No. 2 are a brief narrative and a table of qualification testing that has been prepared by Mr. Larry Lowe of the Berkeley Branch of our Division of New Technology, Materials and Research (formerly TRANSLAB).

These two references allow the Structure Representative a fast and easy way to learn just what the 1992 Standard Specifications require the Contractor to do when splicing reinforcing steel.

QUALIFICATION OF BAR REINFORCEMENT SPLICES

splicing of reinforcing bars may be done by lapping, butt welding or mechanical butt and mechanical lap splicing, at the option of the Contractor, and as permitted by the contract specifications. All of these methods are covered under Section 52-1.08 of the Standard Specifications or in the contract Special Provisions. Design limitations may disallow the use of mechanical bar couplers in tension areas subject to stress reversal. Procedure qualification of both welded and mechanical splicing is required for each contract prior to the splicing of any bars. Qualification requirements for both procedures and welder/operators are covered in Section 52-1.08D. Specific requirements for performing butt welded splices and mechanical butt splices are covered in Sections 52-1.08B and 52-1.08C, respectively. Qualification testing of welding procedures and welders are governed by the provisions of Section 6 of the AWS Dl.4 Welding Code. Standard Special Provisions permit electric resistance butt welding by approved fabricators for continuous hoop reinforcement subject to satisfactory tensile testing and visual inspection. Both welding and mechanical splicing must be performed by qualified welders and operators.

For splicing of spiral reinforcement Section 52-1.08 permits vee-groove welded splices, welded lap splices, or mechanical lap splices. The welding details are shown in the contract plans and therefore require no procedure qualification provided acceptable electrodes are used. Acceptance of welding is based on visual inspection only. Job conditions may dictate the use of lap welds only as approved by the Engineer for other than spiral reinforcement. These lap welds must be qualified in accordance with the provisions of AWS D1.4. TRANSLAB will provide field welding inspection support to the Resident Engineer upon request.

All mechanical splices must be of an approved design. Design approval is granted by TRANSLAB, based on extensive testing and supporting test data supplied by the manufacturer. Design approval does not constitute blanket approval for use on contracts without the specified procedure qualification testing. Experience has shown that individual components of splicing systems may vary from the prototypes originally approved, the operators installing the splices may not be properly instructed and installation equipment faulty. All mechanical butt and mechanical lap splices must be qualified for each contract. Additionally, a certificate of compliance must be furnished in accordance with Section 52-1.08C by the manufacturer of mechanical splice materials.

Job Control tests are required for all mechanical butt and mechanical lap splices in accordance with Section 52-1.08E. These test bars will be made in the field concurrent with the work. The Resident Engineer should identify samples with waterproof marking and provide a Form TL-101 Sample Card to accompany the samples to the testing lab, identifying the sampler as well as his/her telephone number. This applies to all test samples made in the field and witnessed by the Resident Engineer. Job control tests for butt welded splices will consist of radiographic testing of 25% of all joints in accordance with the provisions of Section 52-l.O8F. TRANSLAB will provide assistance in radiographic inspection when so requested.

All test samples are to be furnished and the qualification testing done at the Contractor% expense. TRANSLAB will witness all private laboratory tests and relay immediately all test results to the Engineer. The Contractor must furnish copies of all test records to the Engineer.

Ref: 1992 Standard Specifications
AWS D1.4.92 Welding Code

QUALIFICATION TESTING OF REINFORCING STEEL SPLICES
WELDED AND MECHANICAL SPLICES

TYPE	BAR TYPE	TESTS REQUIRED	REF.SPEC.
Full Penetration Butt Weld (Def. Bar Reinforcement)	A615 Gr.60 Def. A706 Gr. Def.	1. Weld Procedure Test* 2. Welder/Operator Test 3. Radiograph 25% of Jts. Not required for A706 when welded with E8016C3 or E8018C3 electrodes when welded per Fig. 3.2 of AWS D1.4	Sec. 52.1.08B,D AWS D1.4-92 Sec. 52.1.08F
Resistance Butt Welded (Spiral & Hoops) Fabricator must be approved by Translab	A615 Gr. 60 Def. & Plain	1. Tensile test (80 ksi min.) 2. Job Control tests (80 ksi min.) 3. Exempt from RT	Spec. Provisions
Vee Groove/Lap Weld (Spiral Reinforcement)	A61 5 Gr. 60 Plain	Visual inspection of procedure and welder's tests for defects	Sec. 52-1 .08 & contract plans
Mechanical Butt Splice + (Def. Bar Reinforcement)	A615 Gr. 60 Def.	1. Design approval by Translab 2. Procedure test 3. Operator test 4. Job Control Test	Sec. 52-1 .02C 52-1 .O8D 52-1 .O8D 52-1 .O8E
Mechanical Lap splice +	A615 Gr. 60 Def.	Same as for mechanical butt splice except slip test not required for procedure tests	

+ Certificate of Compliance required from manufacturer of mechanical splices.

ALL QUALIFICATION TESTING IS REQUIRED FOR EACH CONTRACT WHERE MECHANICALBAR SPLICES ARE USED. FOR WELDED BAR SPLICES PREVIOUS QUALIFICATIONS FOR WELD PROCEDURES AND WELDERS MAY BE ACCEPTED IN ACCORDANCE WITH PROVISIONS OF AWS D1.4. THE FOLLOWING TEST ARE REQUIRED FOR BOTH MECHANICAL BAR SPLICES AND WELDED SPLICES:

1. Procedure Qualification Test:	Mechanical Splices - Sec. 52-1.08D Welded Splices - Sec. 52-1 .08D, Sec. 6 AWS D1.4
2. Welder/Operator Test:	Mechanical Splices - Sec. 52-1 .08D Welded Splices - Sec. 52-1 .08D, Sec. 6 AWS D1.4
3. Job Control Test:	Mechanical Splices - Sec. 52-1 .08E Resistance Welds - Spec. Provisions Welds (AWS 01.4) - Job control tests not required
4. Non-Destructive Tests(RT)	25% of Full Pen. Butt Welded Joints - Sec. 52-1.08F

NOTE: ALL SPLICING METHODS SHALL BE SUBMIT-I-ED TO THE ENGINEER FOR APPROVAL PRIOR TO THEIR USE. ALL TEST SAMPLES AND TEST RECORDS SHALL BE FURNISHED AT THE CONTRACTORS'S EXPENSE. TEST SAMPLES SHALL BE REPRESENTATIVE OF THE SPLICING MATERIALS USED ON THE WORK. TEST SAMPLES SHALL BE FABRICATED AND TESTED IN THE PRESENCE OF THE ENGINEER. TEST RECORDS WITH SATISFACTORY TEST RESULTS SHALL BE PROVIDED TO THE ENGINEER PRIOR TO THE USE OF ANY SPLICING METHOD.

Ref: Section 52 of 1992 Standard Specifications and AWS D1.4-92

BRIDGE CONSTRUCTION BULLETIN **Approved:** ____________ R P. SOMMLARIVA, Chief Office of Structure Construction	**File: BCM 165-7.2** **REINFORCING STEEL** Date: April 8, 1997 Expires: December 31, 1997 Supersedes:

Subject: Mechanical Rebar Couplers - Specification Clarifications

At the request of METS, the following information is to provide clarification for interpreting the Standard Specifications regarding the testing of mechanical butt splices for reinforcing steel:

- The length of the sample splices

  In accordance with Sections 52-1.08D and 52-1.08E of the Standard Specifications, the length of the sample splices for the operator qualification test and the job control test shall be at least 42 inches (1000 mm) long with the splice at mid-length. Under no circumstances can the length of the sample splices be less that 42 inches (500 mm).

- Definition of the “lot” of field splices represented by a job control test

  Unless amended by the Special Provisions, the “lot” of field splices represented by a job control test is defined in Section 52-1.08E of the Standard Specifications as:

  A. For sleeve-swaged mechanical butt splices, a lot is defined to be 150, or a fraction thereof, field splices that are installed
  1) with the same type and model of mechanical coupler,
  2) on the same size bar, and
  3) with the same bar deformation pattern,
  as the sample splices of the job control test.

  B. For all other mechanical butt splices, a lot is defined to be 150, or a fraction thereof, field splices that are installed
  1) with the same type and model of mechanical coupler and
  2) on the same size bar,
  as the sample splices of the job control test.

  Although there may be other material differenceswithin a lot of field splices, such as the heat treatment lot numbers of the mechanical coupler and the reinforcing steel, the only pertinent factors in determining the “lot” of the field splices for job control tests are the ones listed above.

Example 1: Sleeve-Swaged Coupler Type

Total number of field splices = 150
Type of mechanical coupler = Bar Grip
Size of reinforcing bar = #8 only
Bar deformation pattern = 80 with pattern A & 70 with pattern B

Total number of lots = 2
Total job control tests = 2

This example illustrates that even though there are only 150 splices, there are 2 "lots" of field splices because there are two different deformation patterns. The results would be similar if there are two different types or models of mechanical couplers used, or two different sizes of reinforcing bars. The key is that all three factors must be the same within each "lot."

Example 2: All Other Coupler Types

Total number of field splices = 150
Type of mechanical. coupler = HRC Type 510
Size of reinforcing bar = #8 only
Heat treatment lot numbers of couplers = 8 different heats

Total number of lots = 1
Total job control tests = 1

This example illustrates that even though the mechanical couplers came from 8 different heat treatment lots; there is only 1 "lot" of field splices because the heat treatment lot number is immaterial in dete rmining the number of "lots" for job control tests. The only pertinent factors in this case are the type and model of mechanical coupler used and the sizes of the reinforcing bars.

cc: structure Reps
BCEs
ACMs
HQ Office Staff
Consultant Firms
KPinkerman, METS
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 165-7.4 REINFORCING STEEL**
Approved: Original Signed by Dolores Valls Dolores Valls, Deputy Division Chief Offices of Structure Construction	**Date: October 15, 2002 Expires: July 1, 2003 Supersedes: BCM 165-7.4 Dated July 10, 1998**

Subject: Prequalification List for Steel Reinforcing Couplers

The prequalification list for steel reinforcing couplers is available online at the Offices of Structure Construction web page under 'Field Resources'.
http://oscnet.dot.ca.gov/oscnet/

c: BCR&P Manual Holders
Consultant Firms
R. Pieplow, HQ Construction

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 165-10** **REINFORCING STEEL**
Approved: **Original signed by Dolores Valls** **D. M. VALLS, Chief** **Offices of Structure Construction**	**Date: February 14, 2004** **Expires: None** **Supersedes: BCM 165-10 (1/4/00)**

Subject: Reinforcement Splices

Revisions have been made to Section 52-1.08 (Splicing) of the Standard Specifications. These changes will appear in the Amendments to the 1999 Standard Specifications (placed in all contract special provisions) for jobs advertised after November 6, 2003.

Listed below is a summary of the major revisions made to Section 52-1.08:

1. Reinforcing splices are now designated as either 1) lap splice, 2) service splice, 3) ultimate butt splice. Section 52-1.08 addresses the specification requirements for each of these splice types. The ultimate and service splice requirements are now incorporated into Section 52-1.08.

2. No splice zones will be shown on the contract plans. Splice locations for ultimate and service will be shown on the contract plans. Other splice locations (e.g. lap splices) are determined by the Contractor.

3. Shop produced resistance welds must be made at a Caltrans approved facility. A list of approved fabricators can be obtained from METS. Prior to the fabrication of resistance butt welds, the fabricator must submit a QC manual. Other welding requirements are not required (i.e., Section 8-3 of the Special Provisions).

4. Nondestructive splice testing (radiographs) is still a requirement for welded butt splices (excluding resistance welded butt splices) if the specifications allow for sample splices to be taken from other than the completed lot. (e.g. welded butt splice to extend footing or bent cap reinforcing).

5. Pre-job sample tests are no longer required for ultimate butt splices. In lieu of pre-job tests, the specifications now require the contractor to submit a pre-qualification report for review and approval. The purpose of the prequalification report is to document that the contractor has the means and methods to produce acceptable bar reinforcing steel splices. The contractor's QCM must submit the report to the Engineer for approval. Two weeks are allowed for the review and approval of the prequalification report. The prequalification report shall contain:

c: BCR&P Manual Holders

- Splice material information (the chosen splice type must be on the Departments Approved Product List (see http://www.dot.ca.gov/hq/esc/approved_products_list/)
- Names of operators and descriptions of the positions, locations, equipment and procedures
- Operator certification from the splice manufacturer (valid for 2 years).

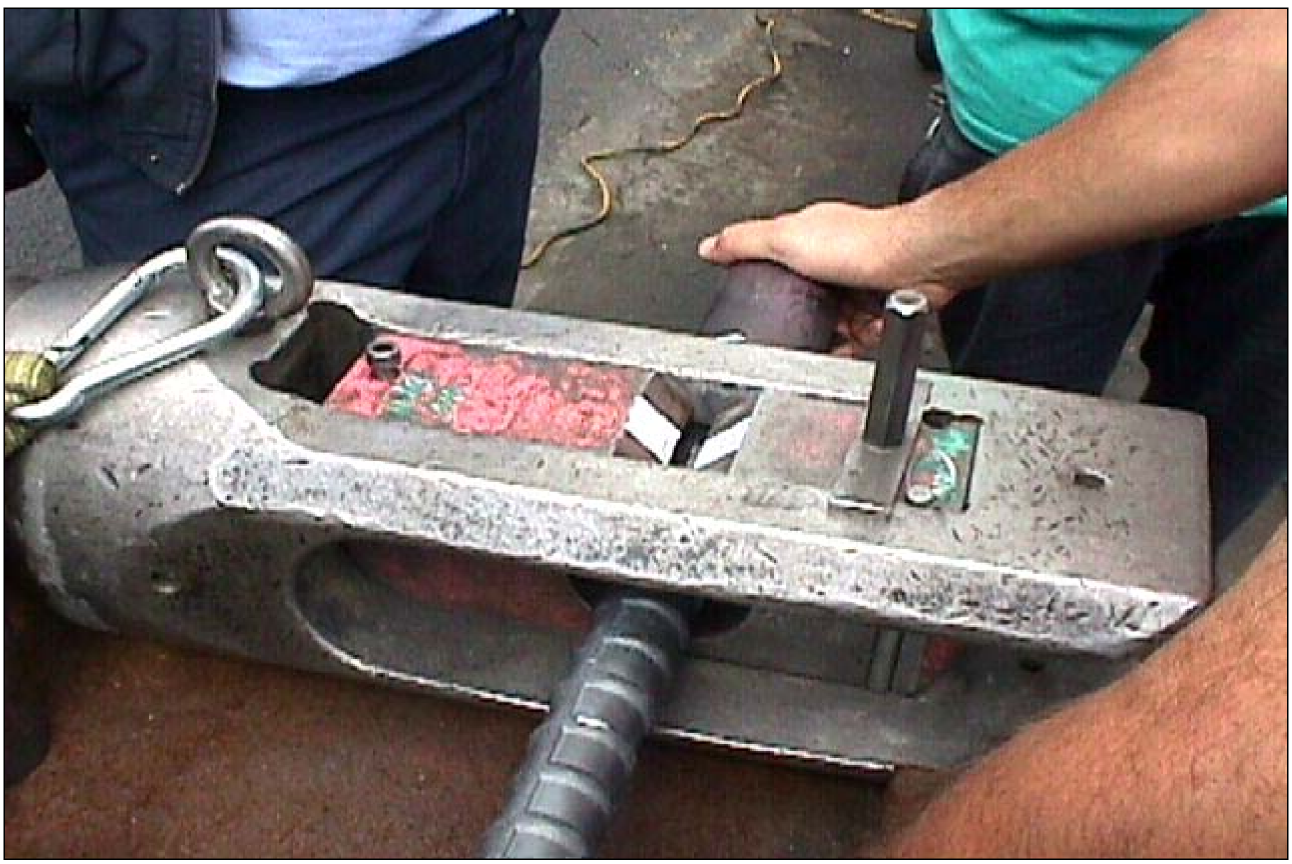
6. The new specification no longer requires Caltrans to witness the Contractor's QC testing of production splices (both ultimate and service). The Contractor is required to notify the Engineer of the testing date and location of the splices one week prior to testing. See Construction Procedure Directive (CPD 04-04) for information related to writing contract change orders on existing jobs to incorporate the above on ongoing contracts.

7. QA testing is now required for service splices. Similar to ultimate butt splices, QA test will be performed by Caltrans. Samples for QA test will be taken on the first production lot and for at least one randomly selected lot every 5 subsequent production tests.

The following attachments will assist the Structure Representative in administrating jobs with reinforcement butt splices. Attachment No. 1 is a summary checklist of the Structure Representative's responsibilities and applicable specification requirements for reinforcement splices. Attachment No. 2 is a flow chart of the ultimate and service splice process.

STRUCTURE CONSTRUCTION

Reinforcing Butt Splice Checklist



Glossary of Terms

Service Splice-	A mechanical or welded splice that meets the current requirements of Section 52 of the Standard Specifications (i.e. tensile strength of 550 MPa (80 ksi) and slip).
Ultimate Butt Splice -	A mechanical or welded butt splice that meets current requirements of Section 52 of the Standard Specifications (i.e. slip test & tensile test which requires the splice to break outside of the splice zone, or within splice zone provided the breaking stress is at least 95% of the ultimate strength of the control bar. In both cases the bar must show visible signs of necking).
Prequalified Products List -	A list of approved mechanical couplers for both service and ultimate type splices. Only splice systems on this list can be used on State jobs.
Splice Prequalification Report -	A report that documents the Contractor's proposed splicing system. The report includes splice material information, names of the certified operators, equipment and procedures. Approval of the Splice report is required prior to performing production work.
Ultimate production splice samples -	Ultimate splice samples that are randomly taken from the completed work. Four samples are selected for every lot of splices and are tested by the approved independent testing facility.
Service production splice samples -	Service splice samples that are prepared in the same manner (i.e., equipment, procedures, position, operator) as the splices incorporated into the final work. Four samples are selected for every lot of splices and are tested by the approved independent testing facility.
Control Bar-	A sample bar that is taken adjacent to the sample splice. All control bars are required to be tensile tested. Control bars are not required for service splice samples.
Quality assurance splice samples -	Ultimate or service splice samples that are prepared in the same manner (i.e., equipment, procedures, position, operator) as the splices incorporated into the final work and then split with a set of production samples for testing at the CALTRANS METS Laboratory. QA tests

	are performed concurrently with the first production lot and at one other randomly selected for every 5 additional production lots (or portion of) thereafter.
Lot of splices	One hundred and fifty (150) or fraction thereof, of the same type and size of splice.
Quality Control Manager (QCM)	Contractor designated person who is responsible for both field and administrative work regarding the quality of all butt splices.
Independent Laboratory	A testing facility the contractor uses to perform the required testing of the sample splices.
Operator certification test	A requirement of the Prequalification Report. Splice operators must be certified. Certification is valid for 2 years.
Resistance Butt Weld	A type of butt-welding commonly used to fuse together column hoop reinforcement. A machine that holds both ends of the hoop together and passes a large electrical current through the bar completes this process. The current creates enough heat to melt the two ends together. This type of welding is not covered by the AWS code and therefore does not require any of the NDT or CWI requirements. The current specification requires the submittal of a Quality Control manual and that the fabricator is approved by the Transportation Laboratory.

I. WHAT TO DO PRIOR TO THE START OF ANY SPLICE WORK

A. The following documents should be reviewed before starting any reinforcing splice work.

- Contract Special Provisions – Amendments to the 1999 Standard Specification, Section 52-1.08
- Contract Plans
- Standard Specifications
  - Section 52 (Reinforcement)
  - Section 6-1.07 (Certificates of Compliance)
- Bridge Construction Records and Procedures Manual
  - Prequalified Products List for steel reinforcement splicing. (found at http://www.dot.ca.gov/hq/esc/approved_products_list/)
  - BCM 165-10 – Reinforcing Splices.
  - BCM 165-7.2– Mechanical Rebar Couplers, Specification Clarification
  - BCM 180-4 – Welding Support from METS
  - BCM 145-16 – Selection of Random Samples
  - BCM 9-1.0 – Locating Reinforcing Steel Splices on As-Built Plans
- California Test 670 – Method of Tests for Steel Reinforcing Bar Butt Splices.

B. Notify METS

As soon as possible, inform your Structure Materials Representative (SMR) that your contract contains ultimate and or service reinforcing splices. Invite SMR to the pre-job meeting. The SMR contact list can be found at http://www.dot.ca.gov/hq/esc/Translab/smbcontacts.htm

C. Review and Approve Contractor's Quality Control Manager (QCM) and Testing Lab(s).

- **Independent Laboratory Requirements**

  The Contractor shall designate in writing a splicing Quality Control Manager (QCM). The QCM will be the sole individual responsible to the Contractor for both field and administrative work related to the quality control of ultimate butt splices. The QCM's duties include inspecting all splice work assuring that completed splices are constructed per the manufacturer's recommendation and the contract specifications, as well as receiving, approving, and submitting all documentation (i.e., correspondence, submittals, reports, etc.) to the Engineer. The QCM cannot be employed or compensated by any subcontractor or by any other persons or entities hired by subcontractors that are providing services or materials for the project. The QCM can be an employee of the prime Contractor.

- **Laboratory Qualifications**

The specifications for both ultimate and service splices require the contractor to select an independent laboratory to test the production splice samples. The Structure Representative needs to review and approve the qualifications of the testing facility. METS should be utilized to assist with reviewing the laboratory's qualifications. The following is a list of specification requirements for laboratories testing ultimate splices.

– Shall not be employed or compensated by any subcontractor, or by other persons or entities hired by subcontractors, who will provide other services or materials for the project.
– Shall have proper facilities including a tensile testing machine capable of breaking the largest size of bar to be tested.
– Shall have a device for measuring the total slip of the reinforcing bars within the splice to the nearest 0.001 inch. This device shall be placed parallel to the longitudinal axis of the bar and shall be able to simultaneously measure movement on both sides of the splice.
– Shall have operators that have been trained in performing the testing requirements of ASTM Designation: A370/A370M and California Test 670.
– Shall have a record of annual calibration of testing equipment. The calibration shall be performed by an independent third party that has 1) standards that are traceable to the National Institute of Standards and Testing and 2) a formal reporting procedure including published test forms.
– An engineer who represents the laboratory and is a registered Civil Engineer in the State of California shall sign test reports.

D. Pre-job Meeting

A pre-job meeting with the contractor should be held to discuss the ultimate and service butt splice specification. The sampling and and acceptance criteria are different for these types of splices. It is important that all parties involved understand the specification requirements.

At the meeting the contractor should have present their QCM, rebar subcontractor, and testing lab representative. If at all possible you should have a representative from METS. A partial list of items to discuss at the pre-job meeting are listed below:

- QCM's responsibility to inspect the lots of splices for conformance with the specifications and manufacturer's recommendations prior to sampling.
- Splice Prequalification Reports, production and quality assurance (QA) sampling and testing requirements.
- How samples of ultimate splices will be selected from a completed lot of splices that have been assembled for the final time.
- Contractor's method of designating and making the lots available for sampling.
- Engineer's method of random sample selection.
- Labeling and shipping of the samples.
- Result reporting, time allowed, and Engineer approval.

E. Splice Prequalification Report.

The specifications require that both service and ultimate splice systems be prequalified, for every job, prior to use. The contractor must select a splice system from the prequalified Products List list. This list is posted at: (http://www.dot.ca.gov/hq/esc/approved_products_list/). If the proposed system is not on the prequalification list, contact METS at (916 227-7253) to verify the latest approved splice systems.

In addition to being on the prequalified list, the contractor must submit for approval a Splice Prequalification Report. The purpose of the Splice Prequalification Report is to document that the contractor has the means and methods to produce high quality splices that conform to the contract specifications. At a minimum the report is required to contain the following:

- Names of the certified operations.
- Descriptions of the positions, locations, equipment and procedures that will be used in the work.
- Operator certification test results.

The contactor performs operator prequalification. Two sample splices for each splice type and bar size, to be used in the work, are prepared by the operator and tested at an independent qualified testing laboratory. If deformation dependent types of splice devices are chosen to be used, two additional samples for each bar size and deformation pattern are required for operator qualification.

II. WHAT TO DO DURING SPLICE PRODUCTION WORK

A. Inspection

During the production of both ultimate and service splices, some of the tasks that CalTrans/OSC personnel should perform are:

- Verify the location of the splices. Note the splice locations on the As-built plans.
- Check the material certification. Certificate of Compliance shall be provided for all material constituting the splice. (i.e. reinforcing steel, coupler)
- Perform visual inspection of the production of ultimate splices.
- Randomly select production and quality assurance sample splices.
- Confirm all test reports meet the contract requirements.

B. Sampling of Production Splices

The sampling procedures and testing criteria are different for ultimate and service type reinforcing splices. Ultimate splices are far more critical to the structures seismic performance. Hence the sampling and testing requirements are more stringent compared to service type splices.

Ultimate splice sampling procedures:

The contractor's QCM will notify the Engineer when a designated lot of splices is complete and has been inspected. Four (4) sample production splices and associated control bars from each lot will be selected by the Engineer for testing. Production sample splices are required to be randomly selected from a completed lot of splices. Selecting from a completed lot means that samples will be removed after final splicing has been made which may or may not entail removing splices from bars after that have been tied in their final location. For example, if the main longitudinal reinforcement of a column was spliced together and assembled on the ground prior to full height erection, the straight bar sample production splices could be selected prior to cage assembly. Similarly, in most cases the selection of production samples for welded hoops could be done prior to cage assembly. However, if a column cage was constructed vertically in its final position, (e.g., column to pier/shaft connection) then the production sample splices would have to be physically removed from the assembled cage. All production sample splices removed from the work must be repaired or replaced. See section II, G. Replacement of Sample Splices.

The intent of the ultimate butt splice specification is to sample splices as close as possible to the in place completed work. Specifying random production sampling from a completed lot of reinforcing splices results in the best possible representation of the actual performance of the as-built splicing system. However, it is recognized that in some cases the structural performance of an ultimate butt splice will be desired but sampling from a completed lot will be inherently risky, not practical or simply not required. The contract special provisions will address these special case splice locations where an ultimate system will be specified but the production test sampling will be modified. Generally speaking, the modified production test sampling procedure for these special cases will mirror the requirements of service splice sampling. Examples of the above special cases are: 1) wet CIDH piles that require longitudinal reinforcing splicing and 2) all ultimate splices in spiral reinforcement (not common in current design practice).

The Structure Representative will normally select production sample splices at the job site, since the sampling must be randomly selected from a completed lot of splices. However, arrangements may be made with METS to sample splices at other locations (e.g., fabricator).

After the splices have been visually inspected samples shall be randomly selected. Refer to BCM 145 –16 "Selection of Random Samples" for guidelines on random sample selection.

Random selection can be accomplished in a variety of ways. The chosen method should be discussed with the Contractor. A few examples of random selection are as follows:

- Selecting 4 numbers out of a hat of 150.
- Use the Excel Spreadsheet Software program and the RANDBETWEEN function.
- Most HP calculators have a function to generate random numbers.

Sample splices and associated control bars for all tests (i.e., prequalification, production and quality assurance) need to be of the following size:

Bar Size	**Length (splice located in the middle)**
No. 25 (No. 8) or smaller	1.5 meter (5 feet)
associated control bar	1.0 meter (3 feet)
No. 29 (No. 9) or larger	2.0 meter (6.5 feet)
associated control bar	1.5 meter (5 feet)

Splice and control bar sample lengths can be shorter. Prior to approving shorter length samples the Structure Representative should discuss and get agreement from both METS and the independent testing laboratories.

Service splice sampling procedures:

The specifications require the contractor to prepare 4 additional service splice samples for every lot of service splices produced. The service splice samples must be prepared in the same conditions as the production service splices. The same operator, equipment, position, and procedures must be used when preparing service splice samples.

C. Quality Assurance Testing

Quality assurance (QA) testing is a requirement of the ultimate and service splice specification. Quality assurance tests are always performed concurrently with the first production test. Subsequent to the first QA test, at least one out of every five additional production tests (or portion of) thereafter will be accompanied by an additional QA test. A random selection method shall be used to designate both QA lots and QA sample splices. Below is a table that illustrates the number of QA tests required for a given amount of splice lots.

No. of Lots	No. of QA tests required
1	1
2-6	2
7-11	3
12-16	4
17-21	5
22-26	6

To obtain samples for the QA test, four (4) additional sample splices, along with the associated control bars, will be made concurrently with the production test samples. These additional sample splices do not have to be removed from a completed lot of splices. The QA samples can be prepared in the same manner as the prequalification samples. In most cases the contractor will simply fabricate an additional four (4) sample splices at the job site. However, in some cases, the contractor may choose to select the QA samples from the completed lot. For example, QA samples for resistance butt-welded hoops could easily be taken from the completed lot before they are tied into the column.

The production and QA sample sets will be split and tested. The Structure Representative will select two (2) of the QA (job control) sample splices and associated control bars, plus two (2) production sample splices and associated control bars and sends them to the METS testing laboratory. The remaining four sample splices (2 QA + 2 production) will be used by the Contractor for the required production test.

D. Tamper Proof Markings and Sample Shipping

To assure that the sample splices are not tampered with, all samples (i.e. pre-job, production, and quality assurance) shall have a tamper proof marking applied to them. Field personnel should apply the marking. Examples of tamper proof markings are: (1) Rubberized paint. This will show any re-gripping or disassembly of the splices (see Figure No. 1) or (2) a digital photo of the splice sent to the lab for comparison. In addition all samples shall be identified as pre-job, production or job control (QA) and be accompanied with a TL-101 form and a testing identification card (see Figure No. 2 for a completed sample).

Both pre-job and quality assurance ultimate splice test samples need to be shipped to the Materials Engineering and Testing Services at 5900 Folsom Boulevard, Sacramento 95819, (916) 227-7251, attention: Byron Berger. The Structure Representative should discuss the method of shipment with METS.

E. Splice Acceptance

Ultimate splice acceptance:

For a lot of ultimate splices to be accepted, the set of four (4) production sample splices must pass both the slip requirement of section 52-1.08C and tensile test as specified in the ultimate splice standard special provisions.
The slip test is done prior to performing the tensile test. Only one of the four (4) splices from the sample set is required to be tested for slip. If this splice does not meet the slip requirements the remaining 3 splices in the set shall be tested for slip. All of these remaining three (3) splices must meet the total slip requirements or the entire lot shall be rejected. If the sample set passes the slip test then the tensile test will be performed.

The ultimate tensile testing requirements are as follows: the sample splices must rupture the bar either 1) outside of the affected splice zone, or 2) within the affected splice zone provided that the sample has achieved at least 95% of the ultimate tensile strength of the associated control bar. The effective splice zone is defined as the portion of the reinforcing bar that has been affected by the fabrication and/or installation of the splice.

In addition to the above tensile test requirements, the sample splices must show visible evidence of bar necking at rupture, regardless if it breaks within or outside the affected zone. Necking is determined by observing a physical reduction in the bar’s cross-sectional area prior to rupture (Figure No. 3 shows an example of bar necking). The necking criteria is an important requirement as it assures that the spliced bars are capable of large strains. This gives the structure the ability to undergo dependable deformations at the plastic hinge zones without experiencing brittle failures.

If three (3) or more splices from the sample set of four (4) meet the tensile test requirements the representative lot shall be accepted. If only two (2) sample splices meet the requirements, an additional 4 sample splices shall be selected from the completed lot and tested. **All** four (4) of the additional sample production splices must meet the above requirements or the lot shall be rejected. If only one (1) of the set of four sample production splices passed the tensile test requirements, the lot shall be rejected. If a lot is rejected, no more ultimate butt splice work shall take place until the QCM completely reviews the Contractor’s quality control process and submits an acceptable plan to correct the situation.

When QA tests are performed the acceptance criteria is identical to the production test. Both the QA and production tests need to pass the requirements in order for the lot to be accepted. As with production testing, if a QA test fails to meet the requirements, no further ultimate splice work shall be permitted until the QCM completely reviews the Contractor's quality control process and submits an acceptable plan to correct the situation. If a QA test fails, only the lot of splices that the QA samples were taken from shall be rejected.

Service splice acceptance criteria:

Service splice samples must pass the slip test and tensile test. The sample set (4 splices) are tested similar to the ultimate splices outlined above except 3 of the 4 sample splices must develop a minimum tensile strength of 550 Mpa (80 ksi). None of the splices in the sample set can have a tensile strength less than 420 Mpa (60 ksi). If only 2 of the 4 sample splices meet the 550 Mpa requirement an additional production test is required. The set of 4 sample splices required for this additional test must be selected from the completed (e.g. in place) lot of splices. All 4 of these additional sample splices must meet the tensile test requirements (550 Mpa). If these requirements are not met, the lot of splices shall be rejected.

F. Test Report

Test reports should be returned on a form that includes all of the information required. OSC 165-01 shows the minimum required information needed from the testing lab. An engineer who represents the testing laboratory and is a registered Civil Engineer in the State of California must sign these reports. Both the QCM and Structure Representative should also sign the reports. All reports must be filed in the job records. In addition, a test report summary sheet should be utilized to help organize and provide easy reference to the records. OSC 165-01 and OSC 165-02 respectively show examples of a test report and summary sheet.

G. Replacement of Sample Splices

Samples that are removed for testing shall be replaced with a splicing system that has been approved for the contract (i.e. a splice that has passed all required prequalification tests). If the production test was successful, the replacement splices, regardless of the preapproved type used, will **not** require testing. If an additional production sample set is required for a lot (i.e. first set of 4 production samples failed) all sample splice locations need to be repaired or replaced prior to selection of the additional set of production sample splices. If the repaired splices are the same types as the original splices, the repaired locations may be included in the original lot to be randomly selected. If a different pre-approved splice system is used for the repair, no additional production tests are needed.

H. Review Time

The ultimate butt splice specifications specify a review time for all splice test reports. To avoid costly job delays, it is important that the Structure Representative respond to the contractor, in writing, within the time allotment given by the specifications. Shown below is a summary table of review times.

Sample Type	Review Time
Prequalification Report	**2 weeks**
Production Sample Tests	**3** working days to review each production test report submitted by the QCM
Quality Assurance	**3** working days upon receipt of the samples by METS **1** extra working day per each simultaneous submittal

III. ITEMS TO BE RECORDED IN THE JOB FILES

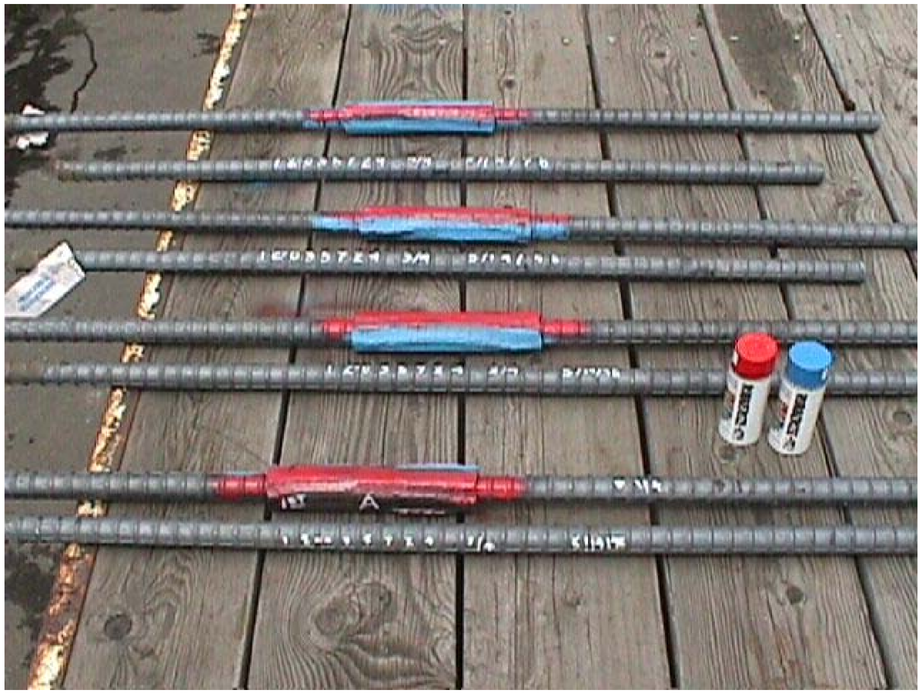
All ultimate splice job documentation shall be filled in Category 37 under the appropriate sub-category headings. Shown below is a list of items to be recorded in the job files:

- Contractor's submitted plan designating the QCM and testing laboratory

- Laboratory Qualification:
  - Record of annual calibration of testing equipment
  - Verification of testing operators ASTM A370/A370M and CA Test 670 formal training.

- Splice test reports:
  - Splice Prequalification Report
  - Production test results
    - Test reports submitted by the QCM
    - Written notification from the Contractor that the splices in the lot conform to the specifications and are ready for testing.
    - Summary record of lots and production test.
    - Records indicating the resolution of any failed test results.
  - Quality Assurance
    - Test results of sample splices sent to METS
    - Summary record of lots and QA test performed. Log shall record that QA tests were performed at specified frequencies (every first production test and one for each five lots thereafter selected randomly).
    - Records indicating the resolution of any failed QA test results.
    - Certificates of compliance

- As-Builts
  - Locate all reinforcing splices on the as-built plans (see BCM 9-1.0).



Production sample splices and associated control bars.
Note the use of rubberized paint as a tamper proof marking system.

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Figure No. 1

TL-101 (REV. 8-76)
STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION

SAMPLE IDENTIFICATION CARD NO. C221379

☐ PRELIMINARY TESTS
☒ PROCESS TESTS
☐ ACCEPTANCE TESTS
INDEPENDENT ASSURANCE TESTS
☐ DIST. LAB
☐ TRANS. LAB
☐ SPECIAL TESTS

SAMPLE SENT TO
☒ HDQTRS. LAB
☐ BRANCH LAB
☐ DIST. LAB

SHIPMENT NO.

AUTHORIZATION NO.

FIELD NO.
DIST. LAB NO.
LOT NO.
P. O. OR REQ. NO.

SAMPLE OF Set of (4) HRC 510 Ultimate Splice & Control Bar

FOR USE IN Column cage

SAMPLE FROM Job Site, Bent 3 Column

DEPTH

LOCATION OF SOURCE HRC Corp, Fountain

THIS SAMPLE IS SHIPPED IN (NO. CONTAINERS) AND IS ONE OF A GROUP OF SAMPLES REPRESENTING (TONS, GALS., BBLS., STA., ETC.)

OWNER OR MANUFACTURER HRC

TOTAL QUANTITY AVAILABLE TEST RESULTS DESIRED ☐ NORMAL ☐ PRIORITY DATE NEEDED

REMARKS Production test for Ultimate Butt Splice Mech. Couplers

COVER ADDITIONAL INFORMATION WITH LETTER

DATE SAMPLED 1/12/00

BY J. W. Lammers TITLE SR

DIST., CO., RTE., P.M. 04-Sol-80-499/531 Bear Creek Bridge, (Br. No. 51-0088)

LIMITS

CONT. NO. 04-111104

FED. NO.

RES. ENGR. OR SUPT. Lammers

ADDRESS 221 First Street, Suite 21, Mytown,

CONTRACTOR Buck's Bridges

MAIL TO SAME DESTINATION AS SAMPLE

Figure No. 2

The picture below shows a passing tensile tested ultimate coupler. Note the bar rupture outside the effective zone and the visible signs of necking.

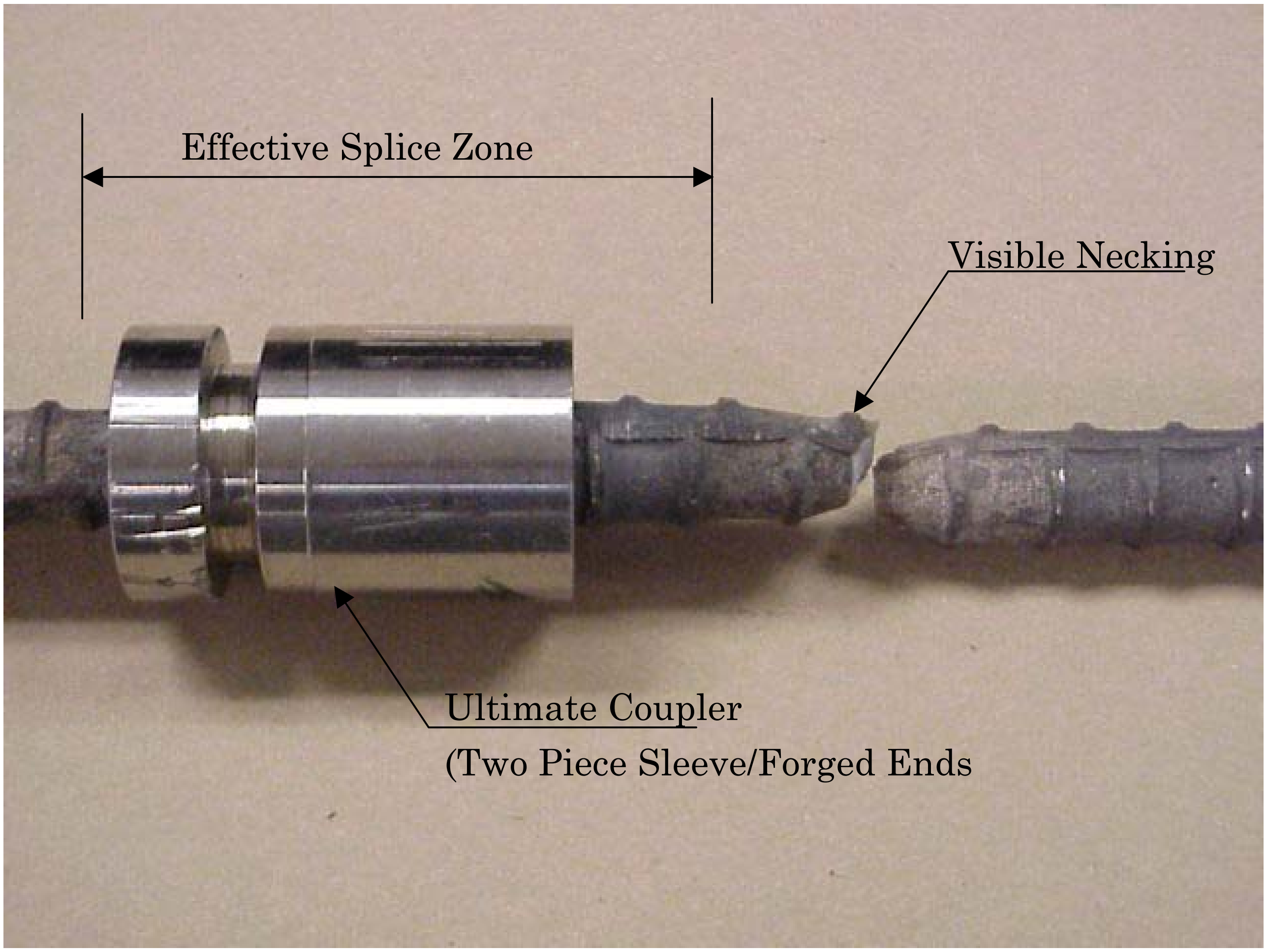


Figure No. 3

Department of Transportation
OFFICES OF STRUCTURE CONSTRUCTION
OSC 165-01

Job Stamp:

Ultimate Butt Splice Test Results

Report No. _______ Date: ____________
Location of Test Sample:____
Lot Number:____________
Date Sampled: ____________
Length of splice: ___________
Splice Type / Bar Size:___________________________ Identification No.______________
Length of affective zone:____________
Notable defects ____

Date Tested:_________________
Testing Laboratory:_____________________
Tester Name: ______________________________

Laboratory Registered Eng. _____
License No. **Exp.**

CALTRANS Rep: _________________________

TEST RESULTS

Sample No.	**Total Measured Slip (mech. only)**	**Splice Ultimate Strength**	**Location of Failure at Ultimate**	**Control Bar Ultimate Strength ***		**95 % of Ultimate (Control Bar)**	**Visible Necking (Yes/No)**	**Pass / Fail**
				1	2			
1								
2								
3								
4								
Comments:								
Recommendation:								

- **min. of one control bar required for each splice**

Quality Control Manager: ______________________________

Structure Representative:__________________

Job Stamp:

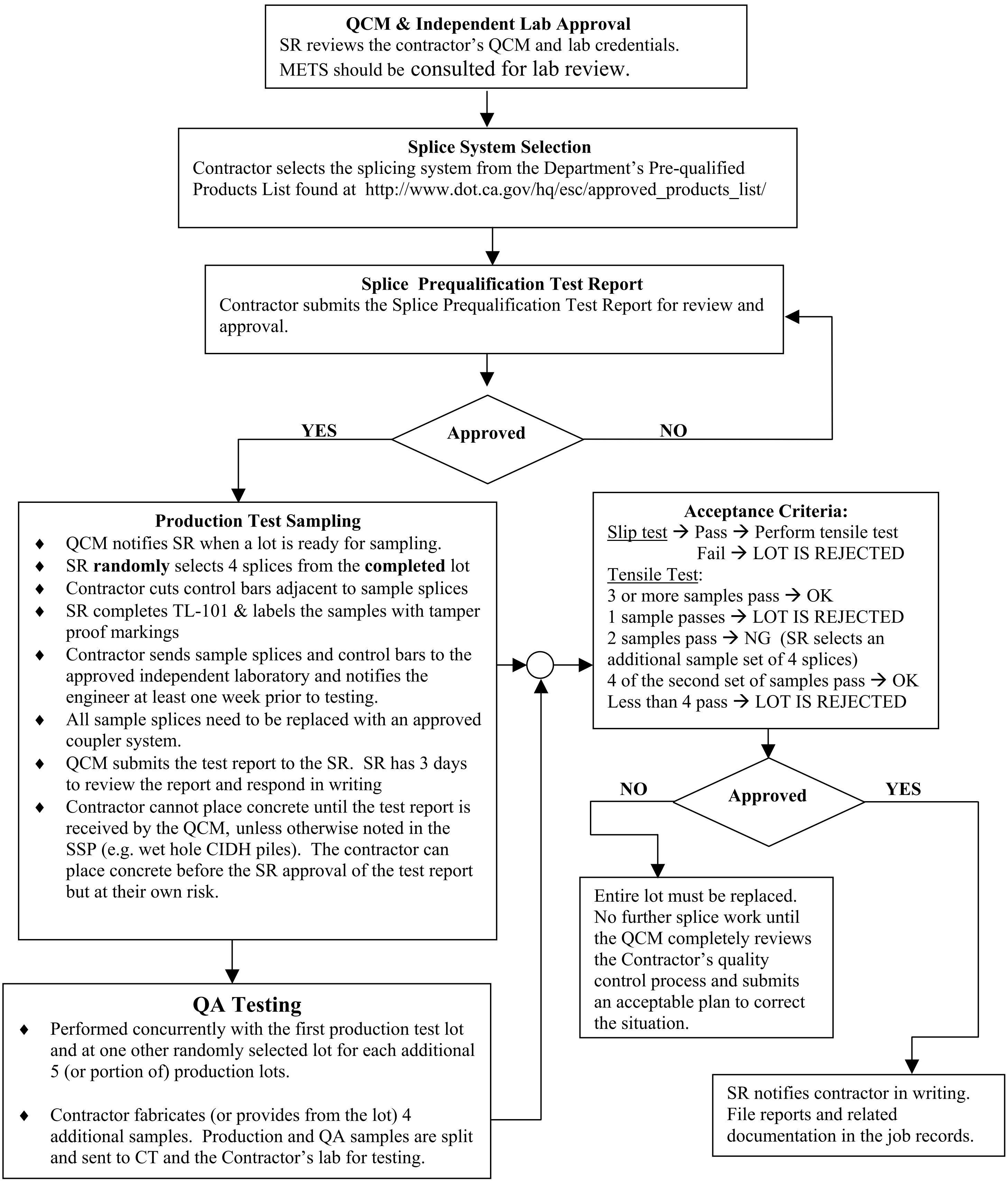
Summary of Ultimate Butt Splice Test Results

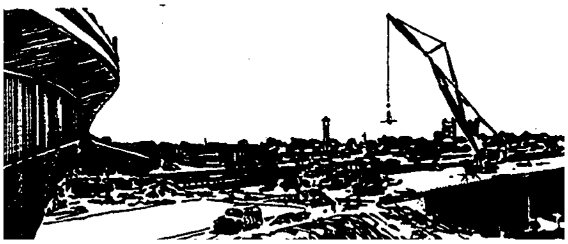
Test No.	Date	Splice Type / Manufacture	Location No.	Lot No.	Pass/ Fail	Comments

Quality Control Manager:________________________

Structure Representative:_________________

ULTIMATE & SERVICE SPLICE FLOW CHART





Volume II

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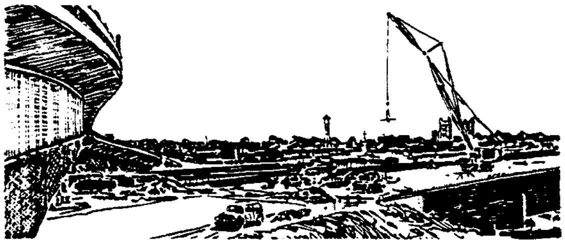
168-2.0	11-15-77	EXPANSION ANCHORS FOR BRIDGE MOUNTED SIGNS
168-3.0	8-05-87	SHOP PLANS FOR SIGNS

Loren L. Krueger

A. P. BEZZONE, Chief
Office of Structure Construction



BOLTED CONNECTIONS FOR OVERHEAD SIGN STRUCTURES

Following is a summary of information which will aid personnel charged with the responsibility of inspecting bolted connections for overhead sign structures.

Unless otherwise specified, all bolts and nuts shall conform to the specifications of ASTM Designation A-307. Also, unless otherwise specified, A-307 bolts should be furnished with commercial quality washers, have hex heads and nuts, and should be "wrench tight". A-307 bolts should be of such length that they extend entirely through the nut (or nuts), but not more than ¼ inch beyond. A-307 bolts in shear shall have not more than one thread within the grip.

Anchor bolts for sign foundations shall conform to the specifications of ASTM Designation A-307. These bolts, washers, and nuts should be galvanized as specified. In addition, the nuts should be retapped after galvanizing to avoid galling and thread stripping. Anchor bolts with properly retapped nuts and clean threads should be tightened sufficiently to prevent removal by hand (wrench tight).

Where high-strength bolts are specified for overhead sign structures, the bolts, nuts and washers shall conform to the specifications of ASTM Designation A-325. ASTM A-325 requires that the bolt head be marked "A-325". In addition, the bolt head may be marked with 3 radial lines spaced 120 degrees apart. High-strength nuts will be marked with the number "2" or "2H", by three equally spaced circumferential lines, or by the letters "D" or "DH".

High-strength bolts used in overhead sign structures may be tightened by any method in order to obtain the required tension.

As indicated in STANDARD SPECIFICATIONS SECTION 55-3.14, a (flat) hardened washer must be installed under the high-strength nut or bolt head, whichever will be turned to tension the bolt, regardless of the method used to tension the bolt, or the type of connection design. Lock washers are not an allowable substitute.

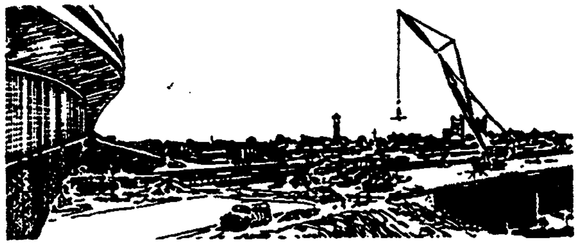
For overhead sign structures, measurement of the bolt tension of field connections shall be by approved direct tension indicators furnished by the contractor. Assembly of high-strength bolted connections for sign structures may be performed with galvanizing or paint on the contact surfaces.

If raised-lug washer-type direct tension indicators are used, one indicator shall be furnished and installed with each bolt in accordance with the following:

Non-galvanized indicators shall be cleaned in a compatible solvent, dried, and dipped in a 1:l solution of Zinc-Rich Primer, Section 91-2.01 and thinner specified therein. Indicators shall be allowed to dry before use.

Washer-type tension indicators shall be installed so that the lugs bear against either a hardened washer or an element of the fastener which is not turned during tightening. After snugging up all bolts of the joint, tightening shall progress from the most rigid part of the joint to the free edges. Bolts shall be tightened until the average gap around the perimeter of the washer is equal to 0.007-inch. Gaps shall not be completely closed. Washer-type indicators shall not be reused,

In connection with tensioning high-strength bolts, the threads of nuts and bolts should be properly prepared to prevent "galling" and excessive friction losses. Therefore, nuts and bolts which are not galvanized ("black") should be clean and dry or lightly oiled. Nuts for high-strength galvanized bolts should be overtapped after galvanizing, and then treated with a lubricant that is clean and dry to the touch. No attempt should be made to tension a high-strength bolt with an un-retapped galvanized nut. The bolt threads will usually gall and strip before the required bolt tension is reached.



BRIDGE CONSTRUCTION MEMO 168-2.0

SIGNS

November 15, 1977

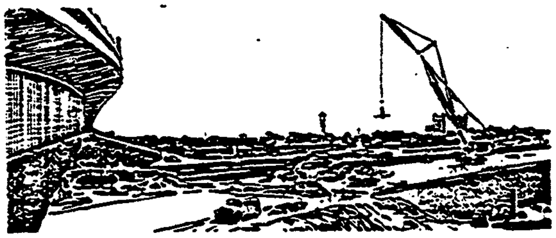
Volume II

EXPANSION ANCHORS FOR BRIDGE MOUNTED SIGNS

Threaded stud bolts must be used when using expansion anchors to attach bridge mounted signs on concrete structures. Headed bolts must not be used in place of threaded stud bolts.

The reason for this is that the use of the headed bolt gives questionable results. as there is no way of confirming whether the expansion anchor has been firmly seated in the concrete, or if it has merely cinched up against the mounting bracket.

For additional information relative to expansion anchors, and for a list of expansion anchors that have been approved by the Transportation Laboratory refer to Bridge Construction Memo 135-5.0.



SHOP PLANS FOR SIGNS

On projects specifying shop plan submittals for signs, plan approval is the responsibility of the Resident Engineer. The Office of Structure Design (OSD) will not review the plans unless unusual details or circumstances exist. Under those conditions, OSD will provide a cursory review upon request to evaluate structural concerns and substantiate design intent. However, final responsibility rests with the Resident Engineer. TRANSLAB should be consulted regarding material questions.

In contracts with "Buy America" clauses, fabricators frequently have difficulty supplying some of the special American steels specified because of the small quantities involved. Some of these steels are more easily obtained from foreign sources and can be used if the quantity or cost does not exceed that allowed by the contract. To comply with the "Buy America" provisions substitutions of steel type may be made by change order if TRANSLAB verifies equivalent structural and galvanizing properties.

BRIDGE CONSTRUCTION MEMO 170-0.0

STRUCTURAL STEEL

July 1, 2001

Sheet 1 of 1

Volume II

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170-4.0	7-1-01	STRUCTURAL STEEL WORKING DRAWINGS

DOLORES M. VALLS, Deputy Division Chief
Offices of Structure Construction

DISCUSSION OF BASIC CONSTRUCTION TERMS AND TOPICS FOR HIGH-STRENGTH BOLTED CONNECTIONS

Discussion of Structural Bolts

There are many types of bolts used for structural applications. Quality for all of these is ensured through compliance with specified American Society for Testing and Materials (ASTM) standard specifications. These national specifications clearly denote specific mechanical properties, chemical composition, and dimensions for each type of fastener.

Where lower strength fasteners are required, ASTM A307, mild steel fasteners and anchor bolts are commonly used. These are usually not preloaded, have a minimum yield strength of 36 ksi, are extremely ductile, and can be welded (when S1 supplementary requirements are specified) and zinc coated.

The main type of high-strength structural bolt frequently specified in Caltrans contracts for steel joints in bridges, overhead sign support structures, and buildings, is designated as ASTM A325, and is available only in a heavy hex headed style and in diameters from ½" through 1-1/2". A325 bolts are almost always specified for major structures, and have a minimum tensile strength of either 105 or 120 ksi, depending on the bolt diameter; the minimum proof load is either 81 or 92 ksi. Because Caltrans wants to insure maximum plastic ductility of fasteners in structural joints in the event of a large earthquake, we specify A 325 fasteners or F1852 tension control (TC) bolts almost exclusively. These can be zinc coated to insure a long life in corrosive coastal environments. While an A490 structural bolt is available, it's lower ductility and inability to be zinc coated make it less desirable for use in coastal regions where long-term corrosion protection is vital and earthquakes are likely to occur.

Where larger sizes of high-strength fasteners or threaded rods having properties identical to those of A325 bolts are required, an A449 series of bolt and rod is readily available. Mechanical properties and chemical composition of this fastener are identical to those of A325 bolts; it is available in a wider variety of sizes, from ¼" to 3" diameters, can be ordered in a number of different head styles, and can be zinc coated.

Another type of high-strength bolt and threaded rod which is quenched and tempered alloy steel, and is called an ASTM A354 is also readily available; it comes in two grades - BC and BD and in diameters from ¼" to 4". Because the tensile strength of Grade BD fasteners may exceed 150 ksi, they cannot be zinc coated. These two grades of fasteners are frequently used for large bolts or rods, where high strengths are required.

Discussion of Various Topics Related to High-Strength Bolting

In the following paragraphs, various topics related to high-strength bolting are discussed:

Types of Connections:

A bolted connection may be designed as either a ***bearing type*** or a ***slip critical*** connection. Caltrans Standard Specifications require that all connections made with high-strength bolts shall be considered as (slip-critical) ***friction-type*** joints, and shall be tensioned as a typical slip-critical joint, unless otherwise designated on the contract plans or specifications. To insure that adequate friction is developed between joint plies, faying (contact) surfaces of all high-strength bolted connections shall be free of rust, mill scale, dirt, grease or any other material foreign to the steel, before assembly. Specifications may require faying surfaces of bolted connections to be coated with either hot-dip zinc coating that has been hand wire brushed prior to assembly, or with an approved inorganic zinc primer prior to assembly.

Bolt Holes:

Bolt holes shall be either punched full size, drilled full size, sub-punched and reamed, or sub-drilled and reamed. Flame cutting of holes is not permitted. Reference Section 55-3.14A, “Bolt Holes” of the Caltrans Standard Specifications, and Table 1 in Section 3(c) of the RCSC Specification. For high-strength bolts, the diameter of standard bolt holes is 1/16” larger than the nominal diameter of the bolt shank.

Thread Stickout:

Determining and purchasing the correct bolt lengths for each different joint is the responsibility of the contractor. The amount of exposed thread beyond the outer face of the nut is called “thread stickout”. After high-strength bolts have been installed and tensioned, the permissible range of thread stickout permitted is from flush to not more than 1/4 inch beyond the outer face of the nut. Note: On TC bolts frequently there are a few partial threads adjacent to the groove where the splined tail breaks off. Therefore for TC bolts, thread stickout shall be measured from the outer face of the nut to the first full thread near the sheared end of the bolt (after the splined end has been sheared off).

Hardened Washers:

According to Section 55-3.14 of the Standard Specifications, one (flat) hardened washer (ASTM F436 or F436M) must be installed under the nut or bolt head, whichever is the element turned in tightening. A maximum of one additional hardened washer may be installed under the non-turning element of the fastener assembly to correct excessive thread stickout. Regardless of the method used to tension the bolt, or the type of connection design, lock washers are not an allowable substitute for hardened washers. Lock washers generally do not have adequate surface contact area, or sufficient corrosion resistance, and due to different steel chemistry and thinner protective coatings, corrode at a higher rate than adjacent steels. If the slope of the exterior face(s) of the connected parts exceeds 1:20 (approximately 3 degrees) relative to the bolt or nut face, a hardened beveled washer(s) meeting requirements in ASTM Specification F436 shall be inserted against each sloped surface.

Snug-Tight Condition:
No matter which of the approved tightening methods is used to tension high-strength bolts, the first step in tightening a joint is the same - bring all plies in the joint in contact by snugging the fasteners. This requires all fasteners in a joint to be brought to a snug-tight condition using a systematic tightening sequence (starting from the center of the joint). “Snug-tight" is defined as the full effort of a person using a spud wrench or a few impacts of a pneumatic wrench applied to the nut. While snugging fasteners, if plies are not initially in contact, care should be taken to avoid bending of the connection parts. Following snugging, all plies in a joint must be in firm contact with each other.

Systematic Tensioning Pattern:
All bolts in a joint need to be tensioned in a systematic manner to produce a consistent even tension in each bolt. The tensioning pattern may be done in a crisscross or alternating fashion, and needs to be systematic to produce an even tension in all bolts. This tightening pattern should be used to bring bolts to the snug condition, and also to their final minimum required tension. In joints having a rectangular or square bolt pattern, bolts must be tensioned, starting at the center (most rigid part) of the joint and proceeding toward the free edges. For joints having a circular bolt pattern, a crisscross alternating pattern is appropriate. Writing a sequential number on each fastener in a large joint is a good way to insure all bolts are tensioned in their correct order, and none miss their turn. To insure that all fasteners are fully tensioned, this final tightening process may require more than one cycle.

Fastener Storage and Handling:
<u>Storage:</u> Regardless which of the approved methods is chosen to tension high-strength bolts, the condition of the fastener components (especially threads on both the nuts and bolts) is critical; all fastener components must be furnished and maintained in good condition until installed and final inspection has been performed. The original lubricant on all fastener components must be kept intact as supplied from the manufacturer, and all fastener components must be stored so that they do not get rusty or dirty. As soon as fastener containers are received at the job site, they must be stored in the original containers and protected from dirt and moisture. Containers should always be covered and be kept off the ground.

<u>Handling:</u> Fastener components from different lots must never be inter-mixed. Only those fastener components that are to be used in one shift are allowed to be removed from containers. Components not used during that shift must be returned to their original containers. The following information must appear on the outside of the shipping/storage containers:

1. Manufacturer’s name and address.
2. Contents (size and numbers).
3. Component lot number.
4. Rotational capacity lot number.

<u>Note:</u> All components of galvanized fastener systems (including bolts, nuts, washers, and DTIs) must be shipped and kept together as an assembly.

Lubrication:

Plain (black) Fasteners: Most plain or "black" fastener components have been heat-treated and all parts are coated by the manufacturer with a thin film of water-soluble (oily) lubricant that can be easily washed-off if exposed to moist elements. Prior to being installed, threads on bolts and nuts shall be oily to the touch, as received by the manufacturer. Should the bolts, nuts, or washers show signs of improper storage, such as rust and dirt accumulation, or absence of original lubricant on the threaded fastener components, this shall be cause for rejection.

Zinc-coated Fasteners: All zinc-coated nuts used on high-strength zinc-coated bolts must be coated by the manufacturer with a lubricant that is clean and dry to the touch, unlike black bolts that are furnished in an oily condition. To make identification easier, a colored dye, or an ultraviolet dye that can be seen with a black light, is required in the lubricant used for all galvanized nuts. No attempt should be made to tension a high-strength, zinc coated bolt whose nut has not been lubricated with a dry lubricant or properly "tapped" oversize. Without the proper lubrication applied on the nut threads and base, the fastener threads can gall, strip or seize, causing the bolt to shear off before the required bolt tension is reached.

Rotational Capacity (RoCap) Test:

At the job site, a rotational capacity test must be done on each lot of both plain and galvanized fasteners to confirm that the nut lubricant, and thread fit and condition as received from the manufacturer will result in proper tensioning without galling or stripping of threads or shearing of the bolt and that the bolt has good ductility. The quality and amount of lubricant and thread fit and condition can vary considerably between various manufacturers and fastener lots, therefore, the use of torque values obtained from charts or tables, or by testing other lots of fasteners is not allowed.

Reuse of High-Strength Fasteners:

Black A325 nuts and bolts may be reused once if allowed by the Engineer. However, neither A490 fasteners nor galvanized A325 fasteners shall be reused after they have been tensioned. Reuse of black A325 bolts and nuts should only be considered if they are in good condition (clean and with lubricant), the bolt threads are not excessively elongated (checked by spinning the nut by hand over the entire length of bolt threads), and each fastener lot is retested and passes the new pre-installation and rotational capacity tests. Once installed, neither TC bolt assemblies nor direct tension indicators (DTIs) may be reused.

Inspecting a Completed Bolted Joint:

Section 55-3.14, "Bolted Connections" of the Caltrans Standard Specifications states, "Bolt tension shall be verified by applying a job inspecting torque to nuts at locations selected by the Engineer. Inspection of each joint should be done as soon as possible, just after tensioning of all fasteners in a joint has been completed. At least 10% of the fasteners in each joint shall be checked. Verification of bolt tension shall be done by the Contractor in the presence of the Engineer and in such a manner that the Engineer can read the torque wrench gage or see gaps around the DTI during checking." The job inspecting torque shall first be determined by the Contractor by testing five fasteners from each lot of bolts according to the procedure detailed in Section 9 (b)(3) of the RCSC Specification. To verify adequate tension in each of the fasteners selected for inspection in a completed joint, a suitable manual torque wrench (dial or digital read out only) is used to apply the job inspecting torque value to nuts (or bolt head, if turned). During the inspection, if any of the nuts turn, then 100% of the bolts in the connection shall be tested, and all bolts found to be under tensioned shall be tightened, and then reinspected.

Definition of Terms Commonly used in High-Strength Bolting

Term's commonly used in high-strength bolting operations and specialized tools need to be clearly understood. The following is a list of terms and tools that are frequently used when dealing with high-strength bolts. Inspectors and construction personnel need to be familiar with these - what they are and how to use them. They include:

Bolt tension calibrator: A machine to measure bolt tensions (i.e., Skidmore-Wilhelm, or Norbar).

DTI (direct tension indicator): A device installed on high-strength bolts to monitor bolt tension. It must conform to requirements in ASTM F959/F959M.

Typical direct tension indicator (DTI)

Electric installation tool for tension control (TC) bolts: An electric tool used to install TC bolts.

Faying surfaces: Contact surfaces between structural plates within a high-strength bolted joint.

Grip length: The total thickness of all plies in a joint, including washers (distance between the underside of the bolt head and the inside face of the nut.

Pre-installation testing: A test series performed on each lot of fasteners, and at the beginning of a shift or job in which the installer demonstrates that with the actual installation equipment and lot of fasteners to be used on the structure, he can properly install them and obtain the proper tension.

Job inspecting torque: A torque value established for each lot of fasteners, and used after a joint has been completed to check that bolts have been tightened to at least the minimum tension.

Match marking: A series of four marks made on the outer surface of a joint, after all fasteners in a joint have been snug tightened to monitor the amount the nut has been turned. Match marking is required if the turn-of-nut tensioning method is used.

Mechanical deposited and hot-dip zinc coating: Two different coating processes where zinc metal is applied to surfaces of fastener components.

Rotational capacity (RoCap) test: A preliminary test performed both by the manufacturer and at the job site on new fasteners to insure that there is proper lubrication on fastener threads and that there is adequate ductility.

Snug tight: The preliminary tightening stage that all fasteners in a joint must be taken to, that produces a tension in each fastener of about 10% of its final tension, and that brings all plies of a joint into firm contact.

Tension Control (TC) fastener: An alternative high-strength fastener system, which includes a nut, washer, and bolt with a splined end. It must conform to requirements in ASTM Specification F1852.

Typical twist-off type TC fastener system

Thread stickout: Amount of threaded bolt tail projecting beyond the outer face of the nut on an installed bolt.

Torque multiplier: A tool used to amplify tightening effort applied to tension (install) or inspect large high-strength bolts.

Torque wrench: A tool (dial or digital type permitted) used to tighten and inspect high-strength bolts.

Volume II

INSPECTION PROCEDURE FOR CHECKING TENSION IN HIGH-STRENGTH BOLTS

Introduction

Following is a brief summary of information that will aid personnel charged with the responsibility of inspecting high-strength bolted connections.

Phases of Inspection

There are three main phases of inspection necessary when high-strength fasteners are installed. These are: 1) Preliminary inspection and testing, 2) Inspection during high-strength fastener installation, and 3) Inspection after high-strength fasteners have been installed.

Phase 1 - Preliminary Inspection and Testing

1. Sampling components and laboratory quality assurance testing:
Fasteners arriving at the job site should be sampled and tested by Caltrans to insure compliance to American Society for Testing and Materials (ASTM) requirements prior to use.

2. Pre-installation testing:
After the satisfactory quality of fasteners is confirmed, the contractor is required to perform pre-installation testing. A calibrated bolt tension-measuring device (Skidmore-Wilhelm or Norbar) is required for this testing. This testing will demonstrate that the contractor has proper equipment and knowledgeable personnel to correctly install high-strength fastener systems being used and can obtain the proper fastener pre-tension for all lots of fasteners to be used. This includes insuring that "snug- tight" tension is correct, impact wrenches and torque wrenches produce the adequate minimum tension, the correct size of calibrated wrench is used (it should take about 10 seconds to fully tension a fastener with a pneumatic or hydraulic wrench).

3. Rotational capacity (RoCap) testing:
This test will verify that the quantity and quality of lubricant and numerous other variables affecting nut factors including thread fit and condition and coating type and thickness will allow fasteners to be tensioned without galling or stripping.

When doing RoCap testing for all lots of fastener systems, a calibrated bolt tension measuring device (calibrated within the last year and traceable to the National Institute of Standards and Technology) shall be used. If fasteners are too short to fit in a bolt tension meter and obtain a full nut, then the short bolt test procedure, as outlined in the current Caltrans Standard Special Provisions shall be used.

Phase 2 - Inspection during High-Strength Fastener Installation

The Inspector shall verify that:

1. The contractor has chosen an acceptable type of high-strength fastener systems as permitted in the contract. Acceptable types may include:
   A. Black bolt (ASTM A325) [with a suitable nut (ASTM A563) and washer (ASTM F436)].
   B. Zinc-coated bolt (ASTM A325) [with a suitable nut (ASTM A563) and washer (ASTM F436)].
   C. Tension control (TC) fastener assembly (ASTM F1852).
   D. Black or mechanically zinc-coated bolt (ASTM A325) [with a zinc-coated Type 325 DTI (ASTM F959), suitable nut (ASTM A563) and washer (ASTM F436)].

2. The contractor is using an approved method of installing high-strength bolts and maintains proper installation technique throughout the project. Approved installation methods include:
   A. Turn-of-nut.
   B. Calibrated wrench [impact wrench (pneumatic, hydraulic, or electric) with positive shut-off system or manual torque wrench - dial or digital only]
   C. Direct tension indicators (DTI's) with black or mechanically zinc-coated bolts.
   D. Tension control (TC) fastener assemblies.

3. All high-strength bolts are installed with a flat hardened washer under the nut or bolt head, whichever is the element turned in tightening. A maximum of one additional hardened washer may be installed under the non-turning element of the fastener assembly so as to prevent the nut from "bottoming out" within the thread transition zone on the bolt shank. (Lock washers are not an allowable substitute).

4. A back-up wrench is used on each fastener to prevent the non-turning element (usually the bolt head) from turning while the fastener is being tensioned.

5. Installation tests have already been run for all equipment and workers involved, and for each different lot of fasteners used. If a different lot of fasteners or installation equipment is used, or new or different installation crewmembers begin work, new pre-installation tests must be conducted.

6. All fasteners in a joint are installed and tensioned at one time. (It is not acceptable to partially install some of the bolts in a joint, or to "stuff" bolts in a joint and let them remain loose for long periods untensioned)

7. All fasteners, no matter which type are used, shall first be taken to a "snug-tight" condition in a systematic tightening pattern, and then fully tensioned in stages using a systematic tightening pattern.

8. Faying surfaces of all plies in each joint and are in firm contact with each other after the members have been brought to a "snug-tight" condition (defined as the full effort of a person using a spud wrench or 12" flex-handle and socket).

9. No short cuts are taken in the proper installation procedure.

10. The fasteners are properly stored after each shift is done and are not allowed to be exposed to degrading elements (especially rain, fog, dampness, dirt, wind, or extreme temperatures).

Phase 3 - Inspection after High-Strength Fasteners Have Been Installed

After all fasteners have been installed and fully tensioned, a final inspection check is done to ensure the job was done properly. This includes 1) a visual check to confirm all plies of a joint are in firm contact, especially around bolts, 2) a check of tension in 10% of the fasteners in each connection (but not less than two) using a torque wrench (dial or digital gage) to confirm that minimum required bolt tension has been attained. This torque requires that a “job inspecting torque” be determined by the contractor for each different lot of fasteners used. A bolt tension calibrator should be used to establish the "job inspecting torque”. Bolt tensions in a joint should be inspected immediately after a joint has been completed. If nuts on any of the bolts checked during the inspection move prior to reaching the job inspecting torque, the remainder of the fasteners in the connection should be inspected and retensioned. Directions for establishing a job inspecting torque value and adjusting tensions in loose bolts are found in paragraph 9(c) of the RCSC Specification (Reference 4 of Attachment No. 3) and shall be followed. Methods for inspecting short bolts are contained in the Structural Bolting Handbook [SBH] (Reference 10 of Attachment No. 3) and require the use of DTIs. Joint seams shall be caulked if needed after fastener tensions in the connection have been inspected and the joint has been approved.

Besides checking bolt tension, the thread stickout should be checked to verify that it is between 0 (flush) and 1/4” beyond the outer face of the nut and that it is the same for all fasteners of similar length. An equal amount of thread stickout in each bolt is an indication that bolt tensions are consistent. Variations in bolt stickout are an indication that some fasteners may be undertensioned, or that joint plies are not in firm contact. Additionally, variations in the thread stickout could indicate that fasteners from different lots have been improperly utilized within the same joint.

It is the contractor's responsibility to provide all required testing equipment and to perform the tests in the presence of the Engineer. If needed, the Division of Structure Construction has bolt tension calibrators and torque wrenches that are available for use by Caltrans personnel for quality assurance inspection.

Attachments No. 1 contain answers to frequently asked questions regarding high-strength fasteners. Attachment No. 2 is a list of specifications and references for high-strength bolting.

COMMON QUESTIONS AND ANSWERS CONCERNING HIGH-STRENGTH FASTENERS

1. Q. What is a Pre-Installation Test (also called an Installation Verification (IV) or Calibration Test)?
   A. The pre-installation tests are performed by the Contractor's personnel using the same installation equipment and witnessed by the Engineer. At least three fasteners from each lot shall be tested in a bolt tension calibration device; if bolts are too short to be installed in such a device, then DTIs and the procedure outlined in the SBH (Reference No. 10 of Attachment No. 3) shall be followed. Rules and required testing frequency are described in Section 8(d) of the RCSC Specification (Reference No. 4 Attachment No. 3). These pre-installation tests will determine the ability of the Contractor's personnel, equipment and procedures used in the actual construction to properly install the same high-strength fasteners used in the structure, according to the approved installation method specified or chosen.

2. Q. What is a Rotational Capacity (RoCap) Test?
   A. This test must be performed by the manufacturer/supplier according to the procedure in the Caltrans Standard Special Provisions. The Contractor is also required to perform the RoCap test at the job site using the same test procedure. This test will verify that the various lots of fastener assemblies when finally ready to be installed at the job site, are capable of withstanding a prescribed nut rotation without failure of the fastener (insures good ductility of fastener), that nuts have been properly lubricated in order to prevent seizing or galling of the threads, and that bolts and nuts are properly tapped and heat treated to prevent thread stripping.

3. Q. Do RoCap tests need to be done on TC bolts, and on fasteners on which DTIs have been installed?
   A. Yes.

4. Q. How many bolt assemblies are necessary for each test required?
   A. Pre-Installation Test: 3 minimum per lot (perhaps checked daily)
   Rotational Capacity Test: 2 minimum per lot
   Job Inspecting Torque determination: 5 minimum per lot (discard 2 test values)

5. Q. May any fastener components which have been used for any tests (including any Pre-Installation, torque/ tension calibration, RoCap, or determination of Job Inspecting Torque) be reused?
   A. No.

6. Q. Why are torque values from torque-tension tables or formulas not permitted to be used to established proper torque?
   A. Each lot of bolts, nuts, and washers is different (amount and type of lubricant, fit and roughness of threads, and thickness, roughness and type of corrosion-protective coating may vary). A standard table or formula relating torque and tension cannot accurately predict the many variables for a particular lot of fasteners; therefore, values chosen from tables or calculated from a theoretical formula are not acceptable. If an emergency situation arises, contact the fastener specialist at Caltrans Division of Materials Engineering and Testing Services (METS).

7. Q. Who determines the bolt length to be used in a connection?
   A. It is the Contractor's responsibility to provide the correct bolt length, unless the Designer has specified the length in the contract documents. Caltrans specifications require that the final thread stickout shall be a maximum of 1/4" and at least flush with the nut face. This insures full bolt thread engagement with the nut, and also provides a maximum number of threads (at least 3 to 5) within the grip length to insure good ductile capacity of the bolt if loaded in extreme conditions.

8. Q. If a bolt is too long, can additional washers be added?
   A. One washer is required to be placed under the nut (or turned end) of the fastener. Caltrans allows only one additional washer to be added (under the unturned fastener end) as a minor adjustment for proper thread stickout.

9. Q. What should be done when fastener holes in joint plies are misaligned?
   A. The Designer should be contacted and address this condition. It may be permissible to ream misaligned bolt holes up to 1/32" over the diameter normally required for a standard hole. Further reaming to permit use of the next size larger fastener may be acceptable if ample spacing, edge distance, and remaining net section are available in the joint and if allowed by the Engineer. Bolt holes shall only be modified by implementing the placement of holes as stated in Section 55-3.14 of the Caltrans Standard Specifications (Reference 1 of Attachment No. 3).

10. Q. Are warped plates allowed in a bolted joint?
    A. Generally, firm contact between plies cannot be attained during the snugging operation, as required, when warped plates or improper fit-up are present in a bolted connection. Gaps around bolt holes and between plies of a friction-type connection are not acceptable. Proper fit-up of a joint prior to bolting is required. Heat straightening and shimming may be possible corrective measures, which can be used to correct warped plates prior to bolting. The Engineer, however, should use prudent judgement as to the acceptability of any material, given the design considerations. The Paragraphs 3.5.1.14 and 3.5.1.15 of the American Welding Society (AWS) Code D1.5 address the general issue of warped plates for mechanically connected joints and splices.

11. Q. What measures should be taken if Contractor does not handle or store fasteners properly?
    A. Section 8(a) of the RCSC Specification requires that fasteners be stored properly. The Inspector at the job site should immediately notify the Contractor if any fastener components are improperly handled or stored, and should document any instances of improper storage or handling in a diary. Proper handling and storage includes: 1) storing fasteners out of the weather in their original containers, off the ground, preferably in a closed building with a roof, 2) removing only as many fasteners from their original containers as can be installed during a work shift, 3) returning unused fasteners to their original containers in protected storage at the end of the shift, and 4) not altering the original lubricant in any way from the way it was in the as-delivered condition. These requirements are all covered in Section 8(a) of the RCSC Specification.

12.Q. What should be done to fasteners that have become dirty or rusty, or have lost their original lubricant?

A. Fastener components that have not been properly stored may have been exposed to moisture, dirt, or dust, and as a result, may have had lost their original lubricant, or become dirty or rusty. Any changes in the original lubricant or thread condition on most fastener components, especially ones such as Tension Control (TC) fasteners, will affect their torque-tension relationship and how they function and may prevent adequate minimum tension from being attained. Fasteners which have become dirty, rusty or whose original lubricant has changed or been altered should be rejected by the Engineer. Whether the rejected fasteners can be restored to a satisfactory useable condition will vary depending on the degree of degradation and damage. If they are deemed salvageable, how they are to be restored to a useable condition and who can do the restoration will vary, depending on the type of fastener, the type of restoration work required, and the facilities available to the Contractor to rework the fastener components. Each case may require the Engineer to assess what facilities and capabilities the Contractor has available and whether he can do a satisfactory job.

Black fasteners are generally easier to clean and relubricate than zinc-coated ones, and in some cases, this operation can be done by the contractor. Light dust or dirt on fasteners can often be removed and fasteners may be relubricated. Rust on fasteners generally results from improper storage and exposure to moisture. The degree of rust damage and the effect of pitting is often more difficult assess and correct. The degree of rust and pitting will determine whether fasteners are salvageable. Light rust on the male threads can often be removed successfully, and fasteners may be relubricated and reused. Moderate to heavy rust that causes heavy pitting usually cannot be corrected and fasteners should be rejected. Rust on the internal threads of nuts is much more difficult to assess or remove; rusty nuts that cannot be thoroughly cleaned or restored should be rejected. Any restoration of damaged fasteners to their original condition and retesting is the responsibility of the contractor. If the Engineer deems that fasteners can be saved, the Contractor is responsible for assuring that the fasteners are thoroughly cleaned and uniformly relubricated, and then for performing additional pre-installation and rotational capacity tests at his expense, to prove the modified fasteners are acceptable.

Often the Contractor is not equipped to perform satisfactory cleaning and relubrication at the job site. Reworking fasteners that have been rejected due to excessive dirt, rust, or lack of proper lubrication requires certain minimum facilities and equipment. These may include a suitable indoor site, equipment and manpower to 1) thoroughly clean the fasteners (i.e., remove all dirt and rust with appropriate cleaning solvent), 2) apply a uniform amount of suitable lubricant similar to what was originally applied to the fasteners, 3) maintain lot integrity of each fastener component requiring cleaning, and repackage each component and remark containers. The Contractor may wish to rework lots of rejected fasteners, but the Engineer needs to judge whether the Contractor is capable of doing a satisfactory job. If the Engineer does not feel that the Contractor is capable of satisfactorily cleaning and relubricating rejected fastener lots, the Engineer should advise him why.

Each component of a black fastener system is originally provided with a water-soluble oil to protect it from rust and to reduce friction when nuts are being snugged and tightened. For zinc-coated fasteners, only the nuts are lubricated with a special dyed, dry lubricant that is clean to the touch.

The type and quantity of lubricant applied by the original manufacturer to nuts on TC fastener systems is very critical and important. Therefore, any lot of TC fasteners that have been rejected for dirt, rust, or improper lubrication should only be reworked, retested, and recertified by the original manufacturer. Any alteration of the original lubricant by anyone other than the original manufacturer voids any certification or warranty made by the manufacturer of a TC fastener system, and should never be allowed. The Engineer should reject TC fastener systems failing to meet any of the required job site tests. The Contractor may return any rejected lot of TC fasteners to the manufacturer for reworking, retesting, and recertification.

The contractor should be aware that some types of lubricant used on fasteners cannot easily be removed from exposed fastener surfaces after installation and prior to painting the bolts. Some lubricants, such as beeswax, are not water-soluble, are extremely difficult to remove, and may require harsh solvents.

Additionally, lubricants should not be sprayed or applied to bolts that have already been installed in a connection, as the lubricant could seep into the faying surfaces of the joint and result in a loss of friction on faying surfaces of a slip-critical joint.

13. Q. Can a Contractor alter (either add or remove) the original lubricant present on fasteners that he received from the manufacturer?

A. No. The original lubricant on the fasteners must not be altered. The manufacturer or responsible party for each fastener system has applied a certain amount and type of lubricant to each fastener in a lot, has tested each lot, and certified that the fasteners comply with all appropriate specifications and ASTM requirements. The original fasteners must be properly stored and maintained to preserve their original condition for all preliminary testing, installation, and tension verification checks on each completed joint. The contractor is not permitted to alter any original lubricant on high-strength fastener systems in any way, either for preliminary testing, or before or during installation. If a particular lot of fasteners should fail any of the preliminary tests required and done at the job site, the Engineer should reject the lot.

14. Q. May one type/grade of high-strength fastener be substituted for another?

A. Generally not. Each grade/type has its own specific material composition, strength and dimensions. Because of smaller head dimensions and shank diameter tolerances, Society of Automotive Engineers (SAE) grades of fasteners (Grades 5 and 8) generally should not be interchanged with ASTM high-strength bolt types. Any request for substitution of a type or grade of bolt different from what was originally specified should be submitted to the Engineer for review prior to acceptance. For further information, contact the high-strength fastener specialist at the Division of METS.

15. Q. If the exterior surface of any steel member is sloped/angled greater than 1:20; can high-strength bolts be used?

A. Yes; however, if the slope of the exterior face of any member exceeds 1:20 (about 2.9 degrees), relative to the washer-faced bearing surface of the bolt or nut face, a hardened beveled washer must be used between the exterior face of the sloped steel part and the bolt head and/or nut to compensate for the excessive slope, and reduce the slope(s) to less than 1:20.

16. Q. May high-strength bolts that were used/tightened once, be reused?
    A. Neither ASTM A490 nor galvanized A325 bolts may be reused. Only plain "black" A325 high-strength bolts should be considered for reuse. Reuse of any black A325 bolts and nuts should only be permitted if the Engineer determines the bolts are in good condition, the bolt threads have not been significantly elongated plastically (this can be checked by spinning the nut by hand over the entire length of bolt threads), and each lot of used fasteners is re-tested and passes the pre-installation and rotational capacity tests. All fastener components used for pre-installation or rotational capacity tests, or for determining job inspecting torques shall be discarded.

17. Q. May TC bolts and/or DTI's be reused?
    A. No. Once installed and fully tensioned or used for any type of testing, they must be discarded.

18. Q. Where should a DTI be installed, which way do the bumps face, and how do I determine if the bolt has adequate tension?
    A. The correct preferred position of a DTI is under the bolt head, with the DTI bumps bearing against the underside of the hardened bolt head. Alternate positions are possible, but only when reviewed and approved by the Engineer. DTI bumps must never bear against any soft steel or any turned component. For bolts to have adequate tension, the gaps on zinc-coated DTIs need to be compressed to 0.005" or less (and also need to be greater than 0). The manufacturer's installation procedure should be followed. For more information, obtain appropriate installation instructions from either DTI manufacturer (see Sheet 10 of 10 of Attachment No. 2), or contact the fastener specialist at the Division of Materials Engineering and Testing Services.

19. Q. Who establishes the job inspecting torque and how is it determined?
    A. The Contractor determines the value for inspection torque by testing 5 fasteners, in the presence of the Engineer, in accordance with Section 9(b)(3) of the RCSC Specification. One high and one low reading are discarded, and the remaining three readings are averaged. The Engineer will record the job torque, determine which bolts in the joint shall be inspected, and witness the Contractor performing the actual checking. The procedure shall be performed in accordance with Section 9(b)(4) of the RCSC Specification.

20. Q. Can a contractor partially install ("stuff") some or all fasteners loosely in a joint with the intent of coming back in the near future and completing his tightening operation?
    A. No, absolutely not! The RCSC Specification [Section 8(A)] clearly prohibits this practice. Only as many fasteners as can be completely installed and tensioned during a work shift can be removed from the storage area. This rule helps prevent fasteners from loosing their lubricant and rusting before the tightening operation and tension verification check has been completed. Occasionally an uneducated or unscrupulous contractor will attempt to do this so that he can speed up his operation. Wise inspectors of course prevent this practice and explain why it is a bad thing to do.

21. Q. Why must hot-dip galvanized faying surfaces be hand wire brushed?
    A. Hand wire brushing is required in order to assure that the galvanized surfaces will have sufficient friction between the plates in contact. Using power driven wire brushes can result in polishing of the surfaces, which would reduce the friction between the surfaces and the capacity of the connection.

22. Q. Why is the thread stickout limited to ¼ inch beyond the face of the nut?
    A. If the thread stickout exceeds ¼ inch, the length of full threads within the grip of the joint is very short, and any elongation that occurs in the bolt during tightening is limited to a very small portion of bolt threads within the grip. Excessive thread stickout reduces the ductile capacity of the fastener during extreme unusual combined tensile and shear loading that might take place during an earthquake. In addition, if thread stickout is extremely large, it is possible that the nut would "bottom out" in the transition zone of the threads during tightening and prior to the full tension of the bolt being achieved. In this case, there may be insufficient tension in the bolt although high torque readings may give a false indication otherwise.

23. Q. What level of inspection is required in order to assure that the bolts have been installed properly?
    A. All stages of bolt installation and tensioning must be witnessed in order to assure compliance with the specifications. It is the responsibility of the inspector witnessing high-strength bolting at the job site to thoroughly understand and enforce Sections 2, 3, and 8 of the RCSC Specification. Verifying that the required final torque has been achieved, without witnessing that the snugging and tensioning operations were performed properly, does not guarantee that, after the joint has been completed, each of the fasteners have the minimum tension required.

LIST OF SPECIFICATIONS AND REFERENCES FOR HIGH-STRENGTH BOLTING:

1. Caltrans Standard Specifications, Section 55-3.14, "Bolted Connections".

2. Standard Special Provisions for high-strength bolting.

3. Project Special Provisions.

4. "**Specification for Structural Joints Using ASTM A325 or A490 Bolts**"**(RCSC Specification)**, Research Council on Structural Connections, American Institute of Steel Construction, Inc., (Allowable Stress Design edition [publication No. S329 (20M596)] or Load and Resistance Factor Design edition [publication No. S345L (30M496)] Chicago, IL, June 3, 1994. Phone No. 1-800-644-2400. ***Available on the OSC Web Site at:***

   http://oscnet.dot.ca.gov/oscnet/

   Note: By reference in the Caltrans Standard Specifications, this RCSC Specification is made a part of all Caltrans construction contracts. The use of high-strength bolts in structural steel connections must conform to requirements in this specification, unless otherwise stated in the contract Standard Specifications or Standard Special Provisions.

5. The following Specifications within the Annual Book of ASTM Standards, Volume 01.08, "Fasteners":
   - ASTM A325 or ASTM A325M, "Structural Bolts"
   - ASTM A563 or ASTM A563M, "Nuts"
   - ASTM F436 or F436M, "Hardened Washers"
   - ASTM F959 or F959M, zinc coated "Direct Tension Indicators"
   - ASTM F1852, "Twist off type TC Bolt Assemblies"

6. The following National Standard titled "Fasteners for Use in Structural Applications", ASME B18.2.6-1996, published by the American Society of Mechanical Engineers.

7. "High Strength Bolts for Bridges", Report No. FHWA-SA-91-031, May 1991, U.S. Department of Transportation, Federal Highway Administration.

8. Division II- Construction, Article 11.5.6, "Connections Using High-Strength Bolts", Section 11 "Steel Structures", of the AASHTO Standard Specifications for Highway Bridges, 16th Edition.

9. AISC Steel Construction Manual.

10. **"Structural Bolting Handbook"**, Steel Structures Technology Center, Inc., 42400 W. Nine Mile Rd., Novi, MI, 48375-4132, (1999 edition) Phone: (248) 344-2910. ***(Contact DSC Headquarters [916-227-8387] to obtain a copy).***

11. "Instruction Manual for Installing High-Strength Bolts with Direct Tension Indicators" (ASTM F959) Inch Series Edition, Turna Sure LLC, 340 E. Maple Ave, Suite 303, Langhorne, PA 19047 (July 1999, 10th edition) Phone: 1-800-525-7193.

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STRUCTURAL STEEL

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Volume II

APPROVED METHODS OF TENSIONING HIGH-STRENGTH BOLTED CONNECTIONS

Introduction

The Caltrans approved methods for tensioning of common high-strength bolt systems consists of two standard methods and two alternative methods. The two standard methods are known as the Turn-of-Nut method and the Calibrated Wrench method. The two alternative methods are known as the Twist Off-Type Tension Control (TC) bolts and the Direct Tension Indicator method. The basic steps for field testing, installation and performing final inspection of the standard methods are very similar to those of the alternative methods.

All fastener systems must pass the required pre-installation test, calibration testing and rotational capacity before being installed in a structure. These tests are performed at the job site by the Contractor and are witnessed by the Engineer. The faying (contact) surfaces of all joint plies must be clean and flat. In many instances, a thin coating of qualified paint or hot-dip galvanized zinc coating may be allowed on faying surfaces. The components to be assembled must fit properly such that the faying surfaces between plies in a joint must have full contact when bolts are installed at a snug condition only. All fasteners in a joint must first be tightened to a snug condition before the final tightening process can begin. In both the snugging and final tightening process, a systematic pattern must be used to tighten each joint, using a crisscross sequence to insure that bolts are evenly tensioned. The final tensioning of A325 fasteners in slip-critical bolted connection must have the following minimum tensions:

Nominal Bolt Diameter	**Minimum Tension Values for A325 Fasteners (kips)**	
(Inch)	**Actual Minimum ***	**1.05 x Minimum ****
1/2	12	13
5/8	19	20
3/4	28	29
7/8	39	41
1	51	54
1 - 1/8	56	59
1 - 1/4	71	75
1 - 3/8	85	89
1 - 1/2	103	108

* Tension values equal to 70 percent of specified minimum tensile strength, rounded to the nearest kip as specified in Table 4, titled "Minimum Fastener Tension for Slip-Critical Connections and Connections Subject to Direct Tension, T_m", of the Research Council on Structural Connections (RCSC) Specification for installing A325 fasteners in slip-critical connections.

** Values are used for calibration and pre-installation testing of all A325 high-strength fastener systems.

Once all fasteners in a joint have been fully tensioned, the joint is inspected. This requires 1) a visual check to insure that plies are in full contact, and thread stickout is in the proper range and is uniform for all fasteners, and 2) the job inspecting torque is applied to 10 % of all fasteners in each joint. Joints should always be inspected immediately after being completed. These same basic procedures are common for all of the approved fastener systems.

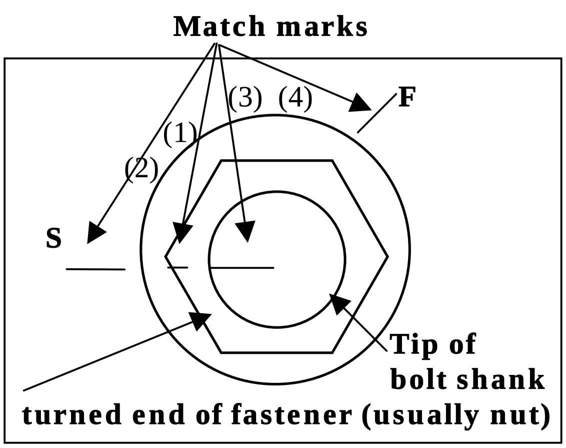
Standard Methods for Installing High-Strength Bolts

The following discussion gives specific information about the two standard methods, Turn-of-Nut and Calibrated Wrench, allowed by Caltrans for installing and checking high-strength bolts:

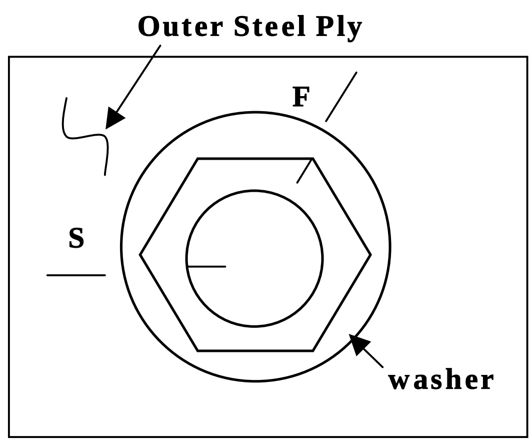
Turn-of-Nut Method

1. First snug tighten all bolts:
When the turn-of-nut method is used, each bolt in a joint must be first brought to a snug-tight condition. At this point, all joint plies should be in firm contact and match marking is done.

2. Match mark all bolts:
When the turn-of-nut tightening method is used to install high-strength bolts, match marking is an important mandatory part of the tightening operation. After snugging, the turned element of all fasteners and the outer plate in the joint are match marked with a felt marker or marking pencil as shown below so that the installer and inspector can see that the nuts have been turned a sufficient amount to adequately tension the fastener. The pictures below show the four initial marks made, and the final position of the marks after tightening has been completed.



Initial position of match marks



Final position of match marks

Note: The two lines on the outer steel ply indicate the start (S) and finish (F) point of the turned element.

In a properly match-marked joint, four marks are made at the turned end of each fastener. These are:

(a). ***A mark on one corner of the nut***. In addition to this mark on one corner of the nut, the outside of the socket used to tighten the nut is usually also marked with a line on its exterior which will be visible during the tightening operation. This mark on the outside of the socket should overlay the hidden mark on the nut corner.
(b). ***A start line, S, put on the outer steel ply*** after all bolts have been snug tightened, and which aligns with the corner mark on the nut.
(c). ***A radial line through the end of the bolt tail***, in line with the start line on the outer steel ply and the nut mark. This radial mark through the bolt tail is important, as it gives a clear indication whether the bolt head turned during tightening (i.e. was properly backed up and kept from rotating during the tightening operation).
(d). ***A finish line, F, on the outer steel ply*** at the appropriate amount of either 1/3, 1/2 or 2/3 of a turn clockwise past the S mark. The location of this (F) mark will vary and depends on the length of the bolt being tightened.

3. Final Tensioning of Fasteners:
The final tightening of the bolt is done as follows:

The socket is positioned so that its exterior mark is aligned with the start (S) mark on the outer steel ply and the mark put on the corner of the nut.The nut is then turned a prescribed amount, depending on the bolt length as shown in Table 5 of the RCSC Specification, until the initial mark made on one corner of the nut lines up with the final mark, F, on the outer ply of the joint. While the nut is being turned, the bolt head (or component of the bolt that will remain stationary) is held with a back-up wrench. The radial mark through the end of the bolt tail should still be aligned with the start mark, S, on the outer ply. If not, this is a clear indication that the bolt head turned during tightening, and the bolt tension may be below the minimum required. This completes the tightening.

The final position of the nut has an allowable tolerance of several degrees with respect to the final mark, F, depending on the size and length of the high-strength bolt. The following are acceptable tolerances:

Bolt Length	Specified Turn	Tolerances Allowed
≤ 4D	1/3 turn (120°)	± 30 °
over 4D but ≤ 8D	1/2 turn (180°)	± 30 °
over 8 D but ≤12D	2/3 turn (240°)	± 45 °

4. Final Check:
Each joint should be inspected as soon as all bolts in the joint have been tensioned. The job inspecting torque check should verify that the bolts in a connection, tightened by the turn-of-nut method, are not below the required minimum tension. Loose bolts may indicate that the bolt heads were allowed to rotate during tightening or the plies of the joint were not in full contact after snug tightening was completed. Therefore, when installing several bolts in a single joint, it is best to snug bolts in at least two tightening stages, and to use a systematic, alternating tightening pattern, starting near the middle of the joint. This process will insure even tension in all bolts when complete.

If the members being joined cannot be brought into firm contact by snugging all bolts, verify that the bolt is the correct length, and that the plies are not misaligned, warped, and do not have burrs and/or irregularities. If plies are misaligned and bolt holes do not line up, the cause of the misfit must be determined and corrected. Further tightening of bolts will generally not correct gaps between plies and around the bolts after they have been snugged, and may result in severely elongated bolts and/or distorted plates.

Calibrated Wrench Method

This tensioning method may be used only when the Contractor's equipment and installation procedures are calibrated daily for each diameter, length, grade, and production lot of bolts.

Torque and/or impact wrenches shall also be recalibrated when significant difference is noted in the surface condition of the bolt or nut threads, or washers.

Calibrated wrenches used for installation shall be set to provide a bolt tension not less than five percent in excess of the minimum tension.

All bolts shall be installed with hardened washers under the turned element, and shall be brought to a snug condition prior to applying the final pretension. Snug tightening shall begin from the middle (or most rigid part) of the connection and progress to the free edges. The tightening operation should be performed such that a systematic (crisscross or alternating) pattern is followed and the same consistent pattern is used for both snugging and final tightening. In some cases, proper tensioning of the bolts may require multiple cycles of systematic partial tightening prior to achieving adequate and even pretension in all the bolts.

When using a torque wrench, the Contractor is required and the Engineer should verify that the torque used on bolts in the structure is consistent with the values determined during the calibration/pre-installation tests done at the beginning of the work shift. In addition, the length of bolt should be checked for compliance to thread stickout limits.

Using a suitable wrench with proper torque capacity for the desired bolt diameter and grade is very important. When the correct size of pneumatic impact wrenches are used, it should take the operator about 10 seconds (after snugging) to achieve the required minimum bolt tension. This condition may result if the time required to tighten a bolt to the minimum required tension is very short (4 seconds or less). It is undesirable to use a wrench which is too powerful for tensioning a particular size or grade of bolt because it can easily result in a fastener whose threaded shank is severely necked down and has been plastically stretched near ultimate capacity; this removes most all of the bolt's residual capacity to stretch and deform plastically without breaking. Using too small, or a worn or broken impact wrench, on the other hand, will not produce the minimum bolt tension required at the recommended 10-second tightening period. Excessive hammering on nuts which results when attempting to tighten a bolt with an inadequate impact tool, (or too little air pressure or air volume) can distort nuts and damage any protective coating, and will still not provide sufficient bolt tension.

The following steps are typical ones used to properly test structural bolts when using a calibrated wrench:

1. The contractor should do calibration/pre-installation testing on a minimum of three bolts, nuts and washers for each diameter, length, grade, and production lot to be used for that day. Testing shall be performed in an appropriate model bolt tension calibrator, according to requirements in the RCSC Specification, Section 8 (d)(2). The contractor must order and use the proper length bolt for a particular joint thickness. Bolts from the same lot that are used in the structure must also be used for verification testing. Additional spacers with the proper center hole diameter must be used to adjust the grip in the bolt tension calibrator, so that two to three threads of stickout is flush with the face of the nut when the nut is finger-tight. Final stickout permitted is between flush and 1/4" past the face of the nut. Appropriate steps, as outlined in the Structural Bolting Handbook should be followed for testing short bolts.

If short bolts are required in the structure and cannot fit into a bolt tension-measuring device, direct tension indicators (DTIs) shall be used to verify adequate tension in the bolts. To determine the appropriate calibrated gap for a particular lot of DTIs, the contractor must furnish longer bolts of the same diameter and grade to be used in the structure, and use them in a bolt tension-measuring device along with DTIs. Once an appropriate calibrated DTI gap is established, the same lot of DTIs shall be used to determine torque or impact wrench setting for the short bolts installed in steel plate shimmed to the appropriate thickness to simulate the actual joint. The short high-strength bolts shall then be tensioned in a simulated joint to produce the same calibrated gap verified with DTIs from the same lot (Reference "Direct Tension Indicator Method) and a torque value read at that gap. The average of the three torque values shall be the installation torque for that lot of short bolts and for that day.

2. First, the bolt must be brought to a snug condition. For the initial snugging, a spud wrench, impact wrench, or bar and socket may be used. The same tools used when installing high-strength bolts in the actual structure shall be used during installation testing.

3. Final tightening should follow one of the two following procedures:

a. Procedure to be used with impact wrench:

(1). Tighten the bolt by turning the nut until the wrench "cuts out". Verify that the tension achieved, as read on the bolt tension-measuring device, is at least 1.05 times the required bolt tension.

(2). Check the degree of turns on the nut to make sure it does not exceed the corresponding tolerance for the "turn-of-nut" rotation. If the amount the nut has been turned has exceeded the maximum rotation allowed, discard the assembly. A new assembly should be tested with the impact wrench torque value adjusted to correspond to the required bolt tension.

(3). The high-strength bolt assembly shall be tested to ensure that the minimum tension is attainable by the installation crew and the tools being used, without exceeding the prescribed rotation.

b. Procedure to be used for torque wrench:

(1). Tighten the bolt by turning the nut until the tension on the bolt is at least 1.05 imes the required bolt tension.

(2). Reading the dial on the torque wrench, measure the moving torque while turning the nut an additional 5 degrees in the tightening (clockwise) direction. This is the torque value that should be recorded.

(3). The average of the three values or the highest acceptable value should be used as the installation torque for this day.

Alternative Methods for Installing High-Strength Bolts

The following discussion gives specific information about the two alternative methods, Twist Off-Type Tension Control (TC) Fastener Assembly and Direct Tension Indicator, allowed by Caltrans for installing and checking high-strength bolts:

Twist Off-Type Tension Control (TC) Fastener Assemblies

All twist-off type tension control (TC) fastener assembly consists of a unique bolt having a splined end, a nut and a hardened washer. The head on the bolt is commonly domed or rounded, but may be manufactured with a hex shape. TC fastener assemblies are produced and shipped by the manufacturers as a precisely engineered and fully tested system. They must comply with requirements in the ASTM F1852 specification. Lubricant types and amounts and machining tolerances may be different from one lot to another, and consequently, the component parts may not be interchanged or altered in any way. Each assembly lot must be used only in the as-delivered, factory-lubricated condition. TC fasteners are installed using an electric wrench having a specially designed planetary chuck. This planetary chuck has dual sockets that engage both the nut and splined tail of the bolt at the same time and turn one relative to the other chuck until the splined tail on the end of the bolt breaks off.

When inspecting a TC fastener installation to ensure a that a quality job is being done, a number of things must be checked: the initial job-site testing of the fasteners must be carefully observed and checked, the installation procedure required by the manufacturer must be reviewed, proper storage of the fastener assemblies out of the elements must be constantly checked, the tensioning operation must be carefully monitored while in progress, and the final tension of at least 10% of the fastener assemblies must be checked using a job inspecting torque. Just verifying at the end of the job that the splined end of each bolt has sheared off is not adequate. This only signifies that at some time during installation, the assembly was subjected to a torque adequate to cause the shearing of the splined tail, not that the final tension in each fastener is adequate.

As with other fastener assemblies, representative samples of TC fastener must be taken from each lot and pre-installation tests run at the beginning of the job. Successful completion of these pre-installation tests will to assure that 1) the installer knows the proper procedure to install the fasteners and follows the manufacturers instructions, 2) the actual equipment he is using to install the fasteners works properly, and 3) the fasteners provide the minimum tension as specified in Section 8 (d)(3) of the RCSC Specification.

When observing pre-installation tests, the following should be verified:

1. A representative sample of not less than three bolts of each diameter, length, grade, and lot shall be installed and tensioned by the Contractor at the job site in a bolt tension calibrator. The Contractor's installer shall demonstrate that each assembly develops a tension not less than five percent greater than the tension required by Table 4 of the RCSC Specification.

2. When testing a TC bolt having a domed head in a bolt tension meter, a flat bushing specifically made for testing the domed tension control bolts must be used under the domed head. These special bushings are not normally furnished as standard parts with bolt tension calibrators. A different size of bushing is required for each bolt diameter being tested and can be purchased through the manufacturer (such as Skidmore-Wilhelm) of the bolt tension calibrator.

3. The TC fastener assembly shall be tested using one flat hardened washer (furnished by the manufacturer of the TC fastener assembly), under the nut (turned element).

Each TC fastener assembly shall first be snugged using the same effort and snugging equipment that will be used on the final structure. During the snugging operation, if the spline breaks off, the bolt shall be removed and the bolt tension at snug tight checked. If the tension at snug tight exceeds 50% of the minimum required tension load, the effort used to snug tighten the fastener should be reduced and new pre-installation tests run.

4. If when running the pre-installation tests, the TC bolts are too short to fit into a bolt tension-measuring device, direct tension indicators (DTIs) must be used to verify the proper tension. First a calibrated DTI gap needs to be determined using three bolts long enough to fit into a Skidmore, tightening each until a load of 1.05 times the minimum preload value has been attained, and then, using tapered feeler gages, determining an average gap value for the compressed DTIs. Once an average calibrated gap value has been determined for three DTIs, the same lot of DTIs shall be used in conjunction with short TC bolts in a simulated joint having the same grip as in the actual structure. When short TC bolts have been installed (tail has been snapped), the DTI gap must be equal or less than the calibrated value determined by using long bolts in a Skidmore bolt tension calibrator. This confirms that the fastener tension is equal to or greater than the minimum required.

Rotational capacity testing is also presently required by Caltrans for this system and for this testing, conventional installation tools should be used (to prevent the splined end from being sheared off).

When tension control fastener assemblies are installed in a structure, the following procedure must be followed:

1. TC fastener systems must always be properly stored out of the weather and maintained in the original condition as supplied by the manufacturer, or else the fastener tension will change and problems will arise.

2. When assembling a TC-bolted connection as with other fastening systems, a TC fastener assembly must be installed in each of the holes of the connection.

3. The bolts shall be systematically snugged (preferably using a conventional tightening tool commonly used for a snugging operation – not an electric TC fastener installation tool) to bring all plies of the joint into firm contact and without yielding or fracturing the splined tails of the fasteners. If the TC fasteners are incorrectly installed and full tensioned in a single continuous operation, they will give a misleading indication to the inspector that all the fasteners are properly tightened. However some of the initially tensioned fasteners may not be. If the plies of the joint are not in firm contact after snugging bolts, then the cause needs to be determined and corrected.

4. Finally during the final tightening (tail snapping operation), each assembly is tightened following a systematic, crisscross pattern starting from the center of each joint.

After installation has been completed and there is any question about whether there is adequate tension in the TC fasteners, the following should be done:

- Uniform and proper thread stickout should be checked. After the spline has broken off, a partially threaded section (approximately 1/8") typically remains; these partial threads at the broken end of the TC bolt are not to be considered as part of the thread stickout. Therefore after installation, the actual length of the projecting bolt stub should extend at least 1/8" beyond the outer face of the nut to a maximum of 3/8".

- The contractor should determine a job inspecting torque value.

- A minimum of 10% of the TC fasteners must be checked using a torque wrench for adequate minimum preload.

Direct Tension Indicators (DTIs)

A direct tension indicator (DTI) is a special device used in conjunction with each high-strength bolt to insure proper tension in the bolt has been attained. DTIs have a number of evenly spaced bumps protruding on one side that are compressed against a hardened surface in a controlled manner. As the bolt is tightened, the bumps are crushed. When they reach a prescribed crushed height (0.005" for bridge and sign structures), the high-strength bolt has been sufficiently tensioned.

Basic steps for field testing of DTIs in a bolt tension calibrator (e.g. Skidmore) are as follows:

1. Test three DTIs of each diameter, grade and production lot, plus three sample bolts, nuts and washers. It is not a requirement that this test be conducted on each separate lot of bolts and nuts. Each DTI, along with sample fasteners, is called a "test assembly".

2. Testing DTIs in a bolt tension calibrator requires the use a special flat bushing and flat hardened washer. The bushing available from the bolt tension calibrator manufacturer (Skidmore-Wilhelm) must be used under the nut (or turned element). DTIs are normally placed under the bolt head, with the bumps bearing directly against the underside of the bolt head (non-turned element).

3. Add spacers and washers under the nut, as necessary, to adjust thread stickout from zero to two threads beyond the face of nut, when the nut is finger-tight.

4. Testing or installing DTIs in a bolt tension calibrator is a two-person operation. While tightening the nut, the bolt head must be prevented from turning.

5. First snug the bolt with a DTI as will be done in the actual structure. In the snug condition, no gap on a DTI may be less than 0.015". Use a 0.015" feeler gage to check for gaps less than 0.015" at snug.

6. Then tighten the nut until the bolt tension as read on the bolt tension calibrator is equal to 1.05 times the minimum required bolt tension. Check how many gaps around the perimeter of the DTI the tapered feeler gage enters. It should enter 1/2 or more of the total number of gaps around the DTI.

7. Continue tightening the fastener until the number of gaps which a 0.005" feeler gage won't enter equals or is greater than that shown in Column 4 of the table on Sheet 10 of 11 in Attachment 2 of BCM 170-2.0. The tension in the bolt as measured by the calibrator must be less than the minimum tensile strength of the bolt.

On the actual structure, verify that bolt heads are held stationary with a back-up wrench when nuts are being turned. In addition, check that all of the bolts in the connection are systematically snugged starting from the center of each joint, and the faying surfaces of all joint plies are in firm contact prior to performing the final tensioning of the bolts.

When installing a DTI, the protrusions shall always be positioned so that they bear against a hardened surface (normally the underside of the bolt head) that must be held stationary as the bolt is being tightened. Before bolts are permitted to be installed in the structure, a representative sample of at least three assemblies, of each diameter, grade, and lot shall be tested in a calibrated bolt tension-measuring device. The test assembly shall include a flat, hardened washer under the turned element. By doing the pre- installation test (also called field test in ASTM F959) the installation crew shall demonstrate that, using the same bolts, snugging and installation tools, and techniques to be used on the actual structure, and compressing the DTI protrusions to an average gap of 0.005", it will achieve a tension no less than 1.05 times the specified minimum bolt tension. This requirement is in Section 8 (d)(4) of the RCSC Specification.

When high-strength bolts are installed in the structure in conjunction with DTIs, the fasteners shall be installed in all holes of the connection and tightened starting from the center (most rigid part) of a joint in a systematic pattern, until all plies of the joint are in firm contact.

When an actual joint is being assembled, the fasteners should be checked to ensure they are uniformly snug. A snug tight condition is indicated by partial compression of the DTI bumps. After snugging bolts, any DTI which has been compressed such that any gap less than 0.015" shall be removed and replaced with a new indicator.

Once all fasteners in a joint have been snugged, the fasteners shall then be systematically tensioned, as was done during snugging. In some cases, proper tensioning of the bolts may require multiple cycles of systematic partial tightening prior to achieving even final bolt tension in order to bring bumps in all DTIs to a uniform gap. When inspected after installation, the minimum number of gaps refusing a 0.005" tapered feeler gage shall be as follows: If all gaps have been reduced to 0 after installation has been completed, the DTI shall be removed and a new DTI and fastener installed.

DTIs should not be used when over-sized holes are present, unless approved by the Engineer and manufacturer of the DTI. If approved, special flat hardened washers must be used.

If a DTI cannot be placed under the bolt head (stationary element) due to unusual field conditions, contact the high-strength fastener specialist at the Caltrans Division of Materials Engineering and Testing Services (METS) for assistance. For DTIs approved for installation under the turned element, special hardened washers with a small inside diameter may be necessary, and can be obtained from the DTI manufacturer.

Attachment No. 1 contains answers to frequently asked questions regarding associated tools and equipment used in high-strength bolting.

COMMON QUESTIONS AND ANSWERS CONCERNING TOOLS USED IN HIGH-STRENGTH BOLTING

Bolt Tension Calibrators:

1. Q. What type of equipment should be used to perform torque and tension checks on high-strength fasteners?

   A. Skidmore-Wilhelm (models MS, M, or ML) or Norbar bolt tension calibrators are assigned to some ACM's to do quality assurance testing.

Typical bolt tension calibrator and steel case

The appropriate model Skidmore should be used, depending on the shortest length bolts to be tested (see chart below).

Minimum Bolt Length (inches) which can be tested in Various Models of Skidmore-Wilhelm Bolt Tension Calibrators

Nominal Bolt Size,	Model of Bolt Tension Calibrator				
	M	ML	MS	H	K (Bench Model)
1/2 – 13	2.25	2.250	2.000		
5/8 – 11	2.50	2.500	2.000	2.750	
3/4 – 10	2.75	2.750	2.000	3.000	
7/8 – 9	3.00	2.750	2.250	3.000	
1 – 8		3.000	2.500	3.250	
1 1/8 – 7		4.750	3.250	5.250	
1 1/4 – 7		5.000	3.375	5.500	5.500
1 3/8 – 6				5.500	5.500
1 1/2 – 6				5.750	5.750
Max. Tension Capacity	80K max.	110K max.	90K max.	170K max.	225K max.
Weight	65 lbs. +				180 lbs.

There are also some older Norbar bolt tension calibrators available; check to make sure equipment has been calibrated within the past year, and is within the required accuracy limits.

2. Q. Are all of the necessary parts available with the basic bolt tension calibrators?
   A. Probably not. To test domed head TC bolts and all DTIs, a special set of flat bushings is required which is not normally furnished with the standard calibrator equipment. Flat bushings are readily available from the Skidmore-Wilhelm Mfg. Co. and are shown below.

Special Flat Bushings Required to Test TC Bolts and DTIs

Nominal Bolt Size, inch	**Model of Bolt Tension Calibrator**					
	M and ML Bushing #/ Approximate Cost		MS Bushing # / Approximate Cost		H Bushing # / Approximate Cost	
1/2	MT-608	$15.00 ea.	MS-608	$30.00 ea.	HT-708	$20.00 ea.
5/8	MT-610	$15.00 ea.	MS-610	$30.00 ea.	HT-710	$20.00 ea.
3/4	MT-612	$15.00 ea.	MS-612	$30.00 ea.	HT-712	$20.00 ea.
7/8	MT-614	$15.00 ea.	MS-614	$30.00 ea.	HT-714	$20.00 ea.
1	MT-616	$15.00 ea.	MS-616	$30.00 ea.	HT-716	$20.00 ea.
1 1/8	MT-618	$20.00 ea.	MS-618	$35.00 ea.	HT-718	$20.00 ea.
1 1/4	MT-620	$20.00 ea.	MS-620	$35.00 ea.	HT-720	$20.00 ea.
1 3/8					HT-722	$20.00 ea.
1 1/2					HT-724	$20.00 ea.

Note: The model MS Skidmore bolt calibrator was developed to test bolts which are shorter than can be tested in the Models M or ML calibrators. In Skidmore-Wilhelm's development/design process, the front plates of the Model MS were made thinner. Skidmore-Wilhelm has pointed out that when testing TC bolts, the thinner MS plates will dish slightly when tensioning the fastener. This dishing may reduce the contact area between the nut face and washer, and may cause a slight increase in the torque required to rotate the nut. This may cause a reduction in tensile load read on the bolt calibrator dial at which the spline shears off. Skidmore recommends ordering and using the thicker plates for the MS model if the tension control fasteners are not meeting minimum tension requirements by 5 to 10%. The Skidmore-Wilhelm Mfg. Co. should be contacted immediately (216-481-4774) if use of their Model MS calibrator results in too low tension values when the spline shears off.

In addition to bushings, spacers of the proper length and inside diameter may also be required when testing long bolts to adjust thread stickout to between 0" (flush) and 1/4".

3. Q. Should Caltrans provide the bolt tension calibrator for use on construction projects?
   A. No. The Contractor is to perform all pre-installation testing, rotational capacity testing, and inspection of completed joints at the job site, and shall provide all necessary tools and appropriate calibrated equipment, including torque wrenches (dial or digital readout only), bolt tension calibrators, impact wrenches, sockets, and torque multipliers to do so. The Engineer is to witness preliminary calibration, testing, select

fasteners to be inspected in a completed joint and witness joint inspection. Caltrans has Skidmore-Wilhelm bolt tension calibrators available for use by Caltrans employees doing quality assurance inspection.

4. Q. How often should the bolt tension calibrator be recalibrated and adjusted and who is qualified to do this?
   A. Recalibration and adjustments should be performed at a minimum annually, by a certified testing facility using equipment that is traceable to the National Institute of Standards and Technology at the Contractor's expense. Required accuracy, after calibration is within ± 2% of the actual load. The following is a list of acceptable laboratories that are qualified to recalibrate and repair bolt tension calibrators:

Manufacturers* and Laboratory Test Facilities for Bolt Tension Calibrators			
1)	Almay Labs 1415 Newton St. Los Angeles, CA 90021 Phone: (213) 746-1555	4)	Advanced Witness Series, Inc.** 910 Bern Ct. #100 San Jose, CA 95112-1237 Phone: (408) 453-5070
2)	Technical Services Group P.O. Box 250 Alameda, CA 94501 Phone: (510) 522-8326	5)	Mountz, Inc. 1080 North 11th Street San Jose, CA 95112 Phone: (408) 292-2214
3)	Skidmore-Wilhelm * Manufacturing Co. 442 South Green Rd. Cleveland, OH 44121 Phone: (216) 481-4774	6)	Norbar Torque Tools Ltd. * Beaumont Road, Banbury Oxon, OX167XJ United Kingdom Phone: 44(0) 1295 270333

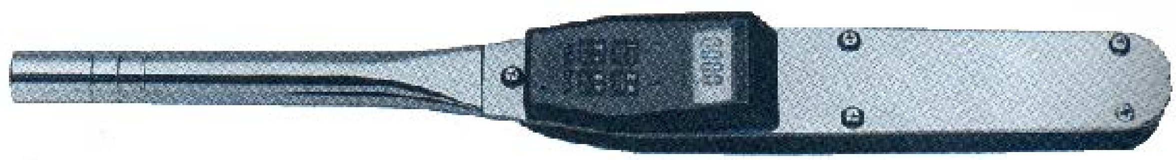
** Local representative and calibration/repair center for Norbar equipment

5. Q. Where can a field Engineer get more information about bolt tension calibrators?
   A. Information and literature can be attained directly from the manufacturers or their representatives (see the listing in Question 4 above).

Torque Wrenches:

1. Q. May the Contractor perform testing or installation of high-strength bolts with a "click-type" torque wrench?
   A. No. Either a dial or digital gauge torque wrench is required to accurately determine installation torque, perform RoCap tests, and determine job inspecting torque for inspection of a completed joint. When performing a RoCap test or determining proper job inspecting torque at the jobsite, the contractor cannot read the particular torque value at a given bolt tension when a click-type torque wrench is used. Also, generally the accuracy of a click-type torque wrench is not adequate to meet Caltrans accuracy requirements.

Dial-type torque wrench



Digital-type torque wrench

Acceptable types of torque wrenches

2. Q. What is the "best practice" when using torque wrenches?
   A. A torque wrench is properly used when the following concepts are followed:

   - Proper installation procedures must be verified and torque values reestablished at least once each working day for each bolt diameter, length, and lot. A hardened washer must be used under the nut (turned end).

   - Always use a proper size torque wrench with adequate torque capacity or use a smaller wrench in conjunction with a torque multiplier, when necessary. Never use a torque wrench that is too small for the job. If the torque limit of a wrench is exceeded, the wrench is generally ruined and cannot be repaired.

   - Operating a large torque wrench is generally a two-person operation - one person to pull on the handle and one to read the dial. The person pulling should use a smooth motion. An extender handle of adequate length should be utilized to reduce effort required to turn nuts and to insure a smooth turning motion (A "jerky" motion generally results when the lever arm is too short). Difficulty attaining adequate tensile loads on large high-strength bolts (i.e., 1-1/8" to 1-1/2") generally indicates the need to use a torque multiplier in conjunction with a torque wrench.

   - It is generally not good practice to use a torque wrench to undo a bolt. A torque wrench is a delicate instrument, and the mechanism of some is not designed to be loaded in the reverse (counter-clockwise) direction. It is much better to use an impact wrench or breaker bar for loosening or unloading tensioned fasteners.

   - When performing RoCap tests, never use a torque wrench to perform the final portion of the test that requires that the bolt be rotated a specified large number of turns. This portion of the test may require a torque level that will severely overload a torque wrench normally used for routine fastener tensioning and cause irreparable damage. A breaker bar or pneumatic wrench in combination with a torque multiplier should be used for this last phase of the RoCap test.

3. Q. Where can I obtain information about or purchase a satisfactory torque wrench?
   A. From one of the companies from the following chart:

Torque Wrench Manufacturers* and Calibrator Companies			
1)	Proto* 14117 Industrial Park Blvd., N.E. Covington, GA 30209 Phone: (770) 787-3800	4)	Armstrong Tools* 5200 W. Armstrong Ave. Chicago, Illinois 60646 Phone: (312) 763-3333
2)	Snap-On* 6632 Fig St., Unit B Arvada, CO 80004 Phone: (888) 762-7972	5)	Advanced Witness Series, Inc. 910 Bern Ct. #100 San Jose, CA 95112-1237 Phone: (408) 453-5070
3)	Mitutoyo* 16925 Gale Ave. City of Industry, CA 91745 Phone: (818) 961-9661	6)	Mountz, Inc. 1080 North 11th Street San Jose, CA 95112 Phone: (408) 292-2214

4. Q. How often should a torque wrench be recalibrated and adjusted?
   A. At least once a year. It must be recalibrated and adjusted more often if dropped/mishandled.

5. Q. Who is qualified to perform the recalibration and adjustments, and what information/certificates are required?
   A. A certified testing facility must have calibration equipment traceable to National Institute of Standardized Testing (NIST) standards and must perform calibration checks and any adjustments on equipment. If equipment cannot be adjusted so that its accuracy is within specifications, it is not permitted to be used.

6. Q. What accuracy is required for a torque wrench to be considered acceptable?
   A. Accuracy of torque wrenches shall be within 2 percent of the actual torque value, with a minimum of 4 verification readings evenly spaced over a range of 20 to 100% of full scale. If there are any questions about the accuracy of the contractor's torque wrench, a copy of the latest calibration check after any adjustments were made should be required.

7. Q. What do the terms *break torque* and *moving torque* mean?
   A. Break Torque: The torque value required to initially start a nut in motion from a stationary position.
   Moving Torque: Torque measured while the nut is in motion.

8. Q. Does Caltrans provide torque wrenches?
   A. Yes, but only for Caltrans quality assurance testing or checking Contractor's values, not for work that is required to be done by the Contractor.

9. Q. When using the calibrated wrench installation method, why is it necessary to determine an installation torque value for each lot of fasteners every day?
   A. Thread conditions (roughness, coating type and thickness, and pitch diameter), the type and amount of lubricant, and storage and weather conditions may be different for each lot of bolts. These variables can have a huge effect on the relationship between torque and tension, especially when using the calibrated wrench method to install high-strength fasteners. Therefore the RCSC Specification requires that certain installation procedures must be performed and appropriate installation torque values

must be determined on a daily basis, using fastener samples from each lot of bolts to be installed in the structure.

Calibrated Wrenches:

1. Q. How often must a calibrated wrench be adjusted and checked to produce proper bolt tension?
   A. At a minimum, the tension value produced by a calibrated wrench must be checked daily at each job shift for each bolt diameter and length to insure the cutoff setting is correct. If a different lot of fasteners is used, or operators, length of air lines, tool being operated, or thread conditions on the fasteners change, the calibration of the wrench must be checked and perhaps recalibrated.

2. Q. Which types of calibrated wrenches are acceptable?
   A. Besides a dial or digital torque wrench, a pneumatic, hydraulic, or electric wrench with an adjustable control unit which can be set to positively shut off at the desired torque is acceptable. A standard impact wrench without a positive cutoff is not acceptable.

Acceptable types of calibrated wrenches

3. Q. What are some equipment variables that effect the final product?
   A. Compressor size and condition, the length, number, and size of air lines, air volume demand of other equipment being operated at same time, condition and size of pneumatic impact wrench, and adjustment settings all can affect torque output of an impact wrench.

4. Q. Why does an impact wrench need to be adjusted and checked so frequently?
   A. Experience has indicated that operator "feel", condition and size of compressors, air lines, air tools, air pressure variations and the number of tools run simultaneously on a single manifold are all variables that account for differences of bolt tension at the snug tight condition, as well as the final tension in the high-strength bolts. The influence of these numerous factors must be checked to insure consistent and accurate bolt tension.

5. Q. Why should it take approximately 10 seconds to fully tension A325 bolts?
   A. A wide variety of brands of calibrated impact wrenches with varying torque capacities are available and appropriate for various sizes of high-strength bolts. When using the correct size of impact wrench with A325 bolts, it takes about 10 seconds (after snugging) to achieve the minimum required bolt tension. Using *too large* of a calibrated wrench for a given size of bolt can result in the threaded shank of the bolt being elongated with the applied stress far exceeding the elastic limit (well into the plastic zone), resulting in very little remaining ductility. Using an impact wrench which is *too small* or *worn* will result in low fastener tension, wasted time and noise

during installation, and excessive hammering on nut, causing damage to any protective coating.

Torque Multipliers:

1. Q. What is this tool and when is it used?
   A. The torque multiplier is a tool that amplifies a small input torque by gear reduction to produce a large output torque. It is commonly used in conjunction with a torque wrench, to reduce the tightening effort needed for testing, installing, and inspecting larger sizes of high-strength bolts. By using a multiplier, less tightening effort can be applied using smaller input wrenches and tools, and smoother and safer tightening operation results. This prevents accidents caused by dangerous but commonly used installation practices of overexertion on a short handled manual torque wrench, or hanging on the end of a long, heavy cheater handle. Also by using a multiplier, permanent damage to smaller torque wrenches which can easily be stressed above their minimum torque rating, can be avoided. An anti wind-up ratchet is a desirable optional feature available on many of the better multipliers.

Acceptable types of torque multipliers

2. Q. Are different sizes available?
   A. Yes. Multipliers are available with various multiplication factors and input and output drive sizes. (E.g., 1:4, 1:5, 1:10, 1:15, 1:25, 1:75, 1:100, etc.)

3. Q. Who manufactures and sells multipliers?
   A. Check the catalogues from following companies: Mountz, Norbar, Proto, Snap-On, and Advanced Witness Series, Inc. Phone numbers for these companies are included in the charts shown in Question 4 on Sheet 3 of 11, Attachment No. 2 of BCM 170-2.0, and in Question 3 on Sheet 5 of 11, Attachment No. 2 of BCM 170-2.0. For further information contact Dan Thomas in the Division of Structure Construction Headquarters or the fastener specialist at the Division of Materials Engineering and Testing Services.

4. Q. How often does a multiplier need to be recalibrated?
   A. At least once a year. When a torque wrench is used in conjunction with a torque multiplier, the two should always be used together with a bolt tension calibrator to determine an accurate input torque/bolt tension relationship for installing and inspecting fasteners.

Electric Installation Tools with combo Spline/Nut Socket for Installing (TC) Bolts:

1. Q. Do all manufacturers of electric installation tools offer the same size of tool?
   A. No, not all electric tools (called Shear Wrench Tools) for installing TC fasteners have the same capacity. Normal electric installation tools require from 11 to 15 inches of working space. Some tools are designed with a right-angled drive for working in tight quarters and have a working clearance of about 7-1/2 inches. In addition, different manufacturers of electric installation tools and sockets for TC fasteners may produce

spline sockets with slightly different internal dimensions. Variations in spline dimensions of TC bolts made by different fastener manufacturers and the actual thickness of zinc coating on the spline section may vary slightly. These conditions can prevent a tool socket from fitting onto the fastener spline. A tight socket clearance or heavy zinc coating on the spline may also prevent easy ejection of the broken spline from the inner socket.

Electric tool for installing TC bolts

2. Q. What variables can effect the satisfactory installation of a TC bolt assembly?
   A. This system is dependent on close manufacturing tolerances (bolt and nut thread dimensions, groove dimensions at tip), steel chemistry, amount and type of nut lubricant, and consistent heat treating all can affect the final bolt tension. Typical steps required with all other high-strength bolt operations, including proper joint fit, snug tightening, tensioning all fasteners evenly in an alternating pattern, are also required when installing TC bolts.

3. Q. Do you need to check the final TC bolt tension with a calibrated torque wrench when fastener installation has been completed?
   A. Yes. Like any normal high-strength bolted connection, joints can be improperly fit and TC bolts can be improperly installed. Plates in joints must flat, should have full bearing after snugging, and all fasteners in a joint must first be evenly snug tightened, and then fully tensioned in stages and in a patterned sequence, or else like any other fastener, some of the TC bolts initially tightened will be loose after the joint has been completed. As required with other tensioning methods, a job inspecting torque must be established and 10% of bolts in each connection (or a minimum of 2 bolts) should be checked for adequate tension immediately after tightening of a joint has been completed.

4. Q. How does one know if TC bolts have been tightened to at least the minimum required tension?
   A. By performing pre-installation tests on each lot, diameter, and length of fasteners, observing that proper installation procedures are followed, and inspecting 10% (or a minimum of 2) bolts in each joint using the appropriate job inspecting torque value.

5. Q. Should all TC bolts in a joint be taken to a snug condition?
   A. Yes. First, all TC fasteners should be first snugged in a connection <u>without snapping off the splined tails</u> before any final tightening is done. Use of a standard impact wrench, torque wrench, or spud wrench is recommended for the snugging operation. If the snugging operation is skipped, and instead, all TC fasteners are fully tensioned without first drawing all plies together, fasteners initially tightened may be loose. If the splined end of a TC bolt has been sheared off, it merely signifies that at some time during the tightening operation, the fastener has been subjected to sufficient torque to cause shearing of the spline. It does not necessarily mean that the final tension is adequate. An uninformed inspector who looks at a completed job may not be aware that just because the tails of all TC fasteners are broken, it does not necessarily mean that all TC fasteners have adequate tension.

<u>Tapered Feeler Gages Required for Measuring Gaps between Bumps on Direct Tension Indicators (DTIs):</u>

1. Q. Why do feeler gages need to be tapered in order to inspect for proper gap in an installed DTI?
   A. Feeler gages used for inspecting DTIs must be tapered and have narrow tips so that they can fit into each gap between closely spaced DTI bumps.

Gage from TurnaSure LLC

Gage from Applied Bolting Technology

<u>Typical tapered feeler gages for inspecting an installed DTI</u>

2. Q. Which thicknesses of tapered feeler gages are used for inspecting DTIs installed on bridge or sign structures: 0.015" or 0.005"?
   A. Two thicknesses of these special tapered feeler gages, 0.005" and 0.015", are commonly used for inspection. The Contractor should have them available and use them during pre-installation testing, installation, testing of short bolts, and determination of job inspecting torques. Caltrans inspectors should also have them handy. The 0.015" feeler gage can be used to inspect DTI gaps after snugging. The 0.005" tapered feeler gage is used to verify adequate DTI crushing during final tensioning for bridge and sign structures. After installation of a DTI is complete, the 0.005" feeler gage tip must be refused in at least 1/2 of the gaps, but all the bumps must not be fully compressed to a "0" gap. A whole set of 26 tapered feeler gages is also necessary when determining a "calibrated gap" for performing pre-installation tests and a job inspecting torque for short bolts (see pp 17 & 18 of the Structural Bolting Handbook).

3. Q. Where can I obtain tapered feeler gages?
   A. The 0.005" and 0.015" tapered feeler gages are available from either DTI manufacturer (see Question 5). Sets of tapered feeler gages (26 leaves) are available from Starrett (model 66T) [phone: 617-249-3551]; Mitutoyo (model 950-242); or McMaster-Carr [phone: 310-692-5911]. The widths of the tapered leaves may have to be trimmed down to match the taper and tip width of the DTI manufacture's gages, so that they will fit between the DTI bumps.

4. Q. How many gaps must refuse the tapered feeler gage for the DTI to be acceptable?
   A. The following chart shows the number of gap refusals required for each size of A325 bolt to insure that the minimum tensile strength has been attained:

Bolt Diameter, inch	Total Number of Gaps on DTI	1.05 x Minimum Bolt Tension, kips	Minimum Gap Refusals For Minimum Bolt Tensile Strength	Minimum Bolt Tensile Strength, kips
1/2	4	13	2	17
5/8	4	20	2	27
3/4	5	29	3	40
7/8	5	41	3	55
1	6	54	3	73
1 1/8	6	59	3	80
1 1/4	7	75	4	102
1 3/8	7	89	4	121
1 1/2	8	108	4	148

5. Q. What companies manufacture DTIs?
   A. In the United States, only the two following companies currently make DTIs:

   1. Applied Bolting Technology Products, Inc.
      P.O. Box 255
      Ludlow, Vermont 05149-0255

      Telephone:....... 802-228-7390
      800-552-1999
      Facsimile:.......... 802-228-7204

   2. TurnaSure LLC
      340 E. Maple Avenue
      Suite 303,
      Langhorne, PA 19047

      Telephone:......... 215-750-1300
      800-525-7193
      Facsimile:......... 215-750-6300

Note: Both DTI manufacturers furnish free installation instructions and tapered feeler gages.

STRUCTURAL STEEL WORKING DRAWINGS

Introduction

The procedure for review and approval of structural steel working drawings is a coordinated effort between Design and Construction personnel. The designer, with input from construction, has the primary responsibility for approval of the working drawings. In the case of projects designed by a Local Agency or Consultant the designer of record, in conjunction with Structure Design and the Design Oversight Engineer, has the primary responsibility for approval of the working drawings. However, the Shop Plan Clerk, in the Structure Design Documents Unit, will furnish the Local Agency Designer or Consultant with two copies of the working drawings.

Working Drawings

Attached is Memo to Designers 12-1 (Attachment No. 1) "Review of Working Drawings - Steel Structures". The Memo to Designers, which is a cooperative effort of Design and Construction, covers the procedures required for review and approval of working drawings including responsibilities of the Structure Representative on a construction project. The procedure for submittal of plans and working drawings is for the Contractor (subcontractor or fabricator) to submit all documents directly to the Office of Structure Design, Documents Unit, Mail Station 9, 1801 30th Street, Sacramento 95816. The original submittals and any resubmittals shall be submitted to the Documents Unit. The Structure Representative should ensure that the contractor submits all documents to the Documents Unit in a timely manner.

The Documents Unit is responsible for administering the working drawing review and approval procedure during all phases of the approval procedure. The Documents Unit maintains a record of all working drawings submitted and distributes copies to the required individuals. This relieves the Structure Representative of the tedious administrative details required in distributing and coordinating the review and approval process.

The responsibility for checking working drawings is shared by the designer of record and the Structure Representative. The Structure Representative shall make all effort to coordinate directly with the designer. The working drawings shall not be returned to the Contractor until the designer of record has discussed and resolved all comments with the Structure Representative. The comments that are returned to the Contractor must be acceptable to both the Designer and the Structure Representative.

The Structure Representative shall ensure that the final working drawings are submitted by the Contractor to the Documents Units, in accordance with Section 55 of the Standard Specifications, prior to the acceptance of the contract. The Structure Representative can verify the submittal of the final working drawings by contacting the Documents Unit at (916) 227-8252.

REVIEW OF WORKING DRAWINGS – STEEL STRUCTURES

Procedure:

The instructions in this Memo apply to working drawings for bridges or other major structures. Working drawings for railings, signs, miscellaneous metal, and other minor items are for the use of field personnel and are not routinely reviewed by the designer. See Article 10 for review of projects designed by Local Agencies and Consultants.

To provide uniform treatment in checking steel working drawings, the following procedures shall be followed:

1. The responsibility for checking working drawings is shared by the designer, the Structure Representative and Transportation Laboratory. Working drawings shall not be returned to the contractor until the designer has discussed and resolved the details with the other reviewers. The comments returned to the contractor must be acceptable to all reviewers.

   A brief memo shall be written by the designer to document controversial decisions or when it is necessary to keep other involved parties informed. For example, a memo is required for any changes or clarification of details in the contract plans. A copy of the memo is to be sent to the Structure Representative and two copies are to be sent to the Transportation Laboratory.

2. When the initial drawings, between six and ten sets, are received, the Documents Unit will forward two sets to the Transportation Laboratory, one set with correspondence to the Structure Representative and the reminder with correspondence will be sent to the design section involved. If less than six sets are received, the Documents Unit shall immediately request the missing sets. The Documents Unit will make this distribution.

   In the event drawings are received for review involving prestressing systems, one of the sets will be forwarded to the Chairperson of the Prestressing Committee for check enroute to the design section involved.

3. The Transportation Laboratory will make the sheets as required and return one set to design.

4. The set of drawings sent to the design section will be the work and file set, that is, it will be marked as necessary in yellow to indicate the checking performed, and in red to indicate any changes required.

5. One of the two sets of drawings sent to the Transportation Laboratory will ultimately be returned to the Contractor. It should not be stamped until all details are resolved between the Structure Representative, the Transportation Laboratory and the Designer and compatible comments transferred to the sheet.

Supersedes Memo to Designers 12-1 dated January 1982

6. Subsequent submittals of working drawings will not be routed out to the Structure Representative or the Transportation Laboratory. If there are any significant changes, the Designer will contact both groups and discuss them before the distribution of prints is made.

7. The Documents Unit will keep the latest set of drawings on file and make them available to the Designer as necessary. When corrected or revised drawings are received, the initial prints will be marked with blue and returned to the design section. All superseded sheets may be disposed of unless a claim or change order is anticipated and there may be a need to reconstruct the history of the project.

8. Members of the structural steel committee may be consulted at any time to assist with technical questions concerning shop practices and procedures.

9. Special Procedure for Structures Carrying Railroads

   a. Specifications require an initial submittal of ten sets of drawings. If less than ten are received, the Documents Unit shall immediately request the missing sets.

   b. From two to four sets are sent to the railroad for their review, with a request that they be *expedited.*

   c. At the same time one set is sent to the Structure Representative, two to the Transportation Laboratory, and the remainder to the design section involved.

   d. The design section is not to return an “approved” or “disapproved” set until all comments, including the railroad’s are received.

   e. When the railroad comments are received, the design section will mark the plans accordingly, or resolve differences as necessary.

   f. The set returned to the Contractor will incorporate both State and Railroad comments.

10. Review of projects designed by Local Agencies and Consultants

   a. Review and oversight of projects involving structures, designed and developed by local agencies or private consultants, is the responsibility of Local Assistance or the Externally Financed Projects Branch.

   b. Occasionally, others may be requested to review technical specialty areas such as walls, railings, earth retaining systems or projects having complex seismic concerns.

   c. Coordination and all plan distribution activities during the design phase will be handled by the Local Assistance or Externally Financed Projects Branch.

   d. Coordination of shop plan submittals by consultants will require special handling by the branch involved.

11. Stamping of Working Drawings

a. Initial Review

1. If they are correct on initial review, the checker shall stamp and date.

Checker shall initial one set of prints only (yellowed check set). This set will be retained in

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OF THE STANDARD SPECIFICATIONS

JUL 7 1989

STATE OF CALIFORNIA
DEPARTMENT OF TRANSPORTATION
DIVISION OF STRUCTURES

the job file by the Documents Unit.

2. If any corrections *whatsoever* are noted, the sheets in error shall be returned for correction. The sheets with corrections shall be stamped and dated:

Checker shall initial one set of prints only. This set will be retained in the job file by the

Prints Reviewed by
State Division of Structures and
RETURNED

JUL 7 1989

FOR
CORRECTION

Documents Unit.

b. Second or subsequent review.

1. If stamped "Returned for Correction" on subsequent review, prints will be handled in the same manner as prints for initial review.

   If only a few minor corrections are made, all sheets needed for distribution must be marked with the same corrections and all stamped:

   Checker shall initial one set of prints only (yellowed check set). This set will be retained in

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   OF THE STANDARD SPECIFICATIONS

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   DEPARTMENT OF TRANSPORTATION
   DIVISION OF STRUCTURES

   MINOR CORRECTIONS SHOWN
   IN RED

   the job file by the Documents Unit.

12. Return to Contractor

    After checking and the discussion with all reviewing agencies is completed, all copies of the Working Drawings shall be returned to the Documents Unit. One of the stamped copies will be sent to the Contractor, and additional copies will be requested as needed for rechecking or for distribution.

13. Distribution of Final Approved Drawings

    a. Working drawing File – one print

    b. Contractor – one print

    c. Structure Representative – two prints (one for his use and one for Contractor's Field Representative).

    d. Transportation Laboratory – two prints (additional prints will be furnished where out-of-State fabricators are involved).

e. Railroads – two prints for each Railroad company involved.

14. Final Disposition of Working Drawings

After completion of a project, the fabricator will furnish 35 mm film of working drawings to the Division of Structures as required by the specifications. These films are checked by the Documents Unit to verify that all required film has been received prior to sending them to file. The films are then filed in roll form by the Documents Unit.

After the films are received the file copies of Working Drawings will be sent to the responsible design section.

15. Guide for Checking Working Drawings

As a means of establishing uniform practice and avoiding omissions, but not as a substitute for common sense, the following outline is submitted as a general guide for checking Structural Steel Working Drawings:

a. Read the Standard Specifications and Special Provisions for the particular job. They may modify the usual procedure. Read the correspondence file; there may have been changes approved by the Office of Structure Construction since the contract was let. Call the Structure Representative to establish a working relationship, and to become familiar with any pending changes or special problems.

b. Changes from the contract plans or specifications, regardless of magnitude, should not be allowed unless they have been discussed and approved by the Structure Representative and Transportation Laboratory. Revisions may be satisfactory structurally but create administrative problems. Changes requiring Contract Change Orders as determined by the Structure Representative need special attention. These change orders could be grouped into two categories:

   1. Those involving changes requested by the State and minor changes requested by the fabricator where there is no question on approval of the change order by both parties. The working drawings can be approved but the note "Contract Change Order to be processed" added to each detail sheet involved.

   2. Those involving controversial changes requested by the fabricator. These should be returned to the fabricator with the note "Request must be made by the Contractor to the Resident Engineer for Contract Change Order." The fabricator may ask that the working drawings be held by design pending such negotiation. Design should not hold any plans without such a request.

c. Review the Contractor's erection procedure to be sure that it will satisfy the assumption for continuity made in design. If the design assumptions are not met, the contractor must submit calculations for revised cambers and stresses. He may be required to increase plate thicknesses or change types of steel.

d. Check to see that all material shown in the working drawings conforms to the size, thickness and steel type shown on the contractor plans or with the requirements of an approved erection procedure.

e. The amount and method of camber should conform to the contract plans or with values computed to accommodate an approved erection procedure.

f. Check the size of all welds. If a welding sequence or procedure other than that shown is proposed, it should be reviewed by the Transportation Laboratory.

g. Check the direction of rolling of plates where specific orientation is required, and the location of butt splices and details of connections not dimensioned on the plans.

h. In general, check only those items listed above. For example, do not routinely make a detailed check of dimensions or the bill of materials.

Philip C Warriner

Guy D. Mancarti

RCA:jgf

Volume II

BRIDGE CONSTRUCTION MEMO 175-0.0

SURVEYING

October 19, 1998

Sheet 1 of 1

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Memo No.	Date	Title
175-1.0	10-19-98	GUIDELINES FOR BRIDGE CONSTRUCTION SURVEYING

RALPH P. SOMMARIVA, Chief
Office of Structure Construction

Volume II

GUIDELINES FOR BRIDGE CONSTRUCTION SURVEYING

Introduction

This memo summarizes Office of Structure Construction policy with respect to the extent of construction surveying to be performed by field personnel.

Although the primary purpose of this memo is to establish specific guidelines and thereby ensure uniformity in stakeout procedure, we recognize that no specific staking policy can be applied in all situations. Of necessity, surveying and stakeout techniques as well as the actual amount of surveying performed must be left to the discretion of the field engineer. To ensure statewide uniformity, however, all field personnel will be expected to conform as closely as possible to the policy as established herein.

Specification References

Section 5-1.07 of the Standard Specifications provides in part as follows:

> “Stakes or marks will be set by the Engineer as the Engineer determines to be necessary to establish the lines and grades required for the completion of the work specified in these specifications, on the plans and in the special provisions."

General Policy

As a general policy the Office of Structure Construction will interpret Section 5-1.07 of the Standard Specifications as provided herein.

Line and grade reference points will be set sufficiently close to the working area to enable the Contractor to accomplish their work using those tools which are normally associated with the work of a bridge construction crew. Such tools include string or wire lines, plumb bobs, carpenters' levels, 15 m (50’) steel tapes, etc.

While it is recognized that a reasonable and prudent contractor should have surveying instruments on the job for their use in constructing the project, it is not required by our policy.

Specific Policy

Within our general policy, certain surveying procedures are subject to specific policy as further described herein. To ensure uniformity, procedures covered by specific policy must be followed on all projects.

1. Establish bent, abutment and wingwall reference points in accordance with the following:

   a Set a minimum of two reference points on each side of the footing.

   b Show distance to principal intersecting control line only. Do not reference intermediate points such as edge of footing, centerline of footing, etc.

   c Set inside reference points sufficiently close to the work that so contractor personnel in accordance with our general policy may use them.

   d Set elevation on inside reference point. Do not show cut to bottom of footing, top of footing, etc.

2. Do not stake the location of individual piles. Do not establish individual pile cutoff elevations.

3. Do not perform any survey work in connection with falsework construction.

4. Set elevation points at the bottom of walls, abutments and columns on the footings so that the Contractor is able to measure up and set their pour strips. Top of column, abutment and wall elevations should be provided to the contractor after they have been calculated and checked either on a grade sheet or on a profile plot with sufficient elevations for the contractor to set their pour strips.

5. Establish edge of deck line at 3 m to 6 m (10 to 20 foot) intervals on soffit forms with a transit. Establish sufficient grades at various places on the soffit forms to allow the Contractor to grade it with string lines or other similar means. Deck elevation points are to be set in accordance with Section 51-1.17, “Finishing Bridge Decks”, of the Standard Specifications.

6. Reference superstructure grades to top of deck only. Do not show cuts or fills to intermediate elevations.

7. Survey for control of barrier rail line and elevation should be performed in accordance with the following:

   a On straight bridges, establish line with a transit and set points at 3 m to 6 m (10 to 20 foot) intervals. On curved bridges, lines may be set from the edge of deck or by transit at the engineer's option.

   b On all bridges, take as-built deck elevations along (or near) rail centerline. Plot profile and make any necessary adjustments to correct for camber, deck irregularities, etc. Grades for fills, as necessary, should be provided to the contractor.

   c Final check should be by "eyeball" to ensure a pleasing appearance in the final product.

8. Bridge deck contour plans should be made available to the Contractor for their use.

Miscellaneous

The Standard Specifications require the contractor to carefully preserve the stakes and marks set by the engineer and make the contractor responsible for the cost of necessary replacement or restoration of stakes or marks which in the judgment of the engineer were carelessly or willfully destroyed by the contractor's operations. Any assessment of restaking charges should be done in accordance with District policies and procedures.

What the contractor can expect in the way of staking and how much notice is expected from them should be discussed at the pre-job meeting.

OFFICE OF STRUCTURE CONSTRUCTION *Bridge Construction Records and Procedures Manual*	B98-05
BRIDGE CONSTRUCTION BULLETIN **Approved:** ________ FOR **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 180-0** **WELDING** **Date: February 20, 1998** **Expires: None** **Supersedes: BCM 180-0.0**

Subject: TABLE OF CONTENTS FOR SECTION 180

Due to the volatile nature of the welding topic, this Bulletin will summarize the titles of the Bulletins that should be in your Bridge Construction Records and Procedures Manual Section 180, Welding.

BCM	Date	Title
180-1.0	12-02-96	FIELD WELDING OF STRUCTURAL STEEL
180-2	12-16-96	CHECKLIST FOR WELDING REINFORCING STEEL, STEEL COLUMN CASINGS, AND STEEL PILES
180-3	12-16-96	REINFORCING STEEL WELDING AND TESTING
180-4	08-01-97	WELDING SUPPORT FROM METS
180-5	12-01-96	WELDING SUPPORT FROM METS FOR PROJECTS REQUIRING RADIOGRAPHS
180-6	06-06-97	REVISE COLUMN CASING WELD BACKING PLATE THICKNESS
180-7	07-15-97	INSPECTION OF WELD BACKING PLATE FOR COLUMN CASING
180-8	02-06-98	RESISTANCE BUTT WELDS - SPECIFICATION CLARIFICATIONS
180-9	02-20-98	REVIEW OF CONTRACTOR'S QUALITY CONTROL PLAN (QCP) FOR FIELD WELDING

c: BCR&P Manual Holders
Consultant Firms
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN **Approved:** ____________ FOR **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM 180-1.0** **WELDING** **Date: December 2, 1996** **Expires: November 30, 1997** **Supersedes: BCM 180-1.0 (3-30-87)** **BCM 180-2.0 (3-30-87)** **BCM 180-3.0 (330-87)**

Subject: Field Welding of Structural Steel

There have been recent developments and problems associated with the workmanship on structural steel contracts and welding of structural steel members. This Bulletin provides an overview of the specificationrequirements, and provides a basic checklist of items to review before, during and after structural steel welding work. A contact list for welding support from the Materials Engineering and Testing Service (METS) is provided under Construction Bulletin, BCM 180-4.0, which is forthcoming within next month.

Work on Structural Steel members shall be governed by the requirements of the Contract Special Provisions, Standard Specifications, Section 55 "Structural Steel", Standard Specifications Section 75, "Miscellaneous Metal" and the most current publication of ANSI/AASHTO/AWS D1.5 "Bridge Welding Code" or ANSI/AASHTO/AWS D 1.1, "Structural Welding Code". The procedure for the handling of Structural Steel Working Drawings is found in Bridge Construction Memo 170-3.0.

Reference copies of Bridge Welding Code, AWSD1.5-95, and Structural Steel Welding Code, AWS D1.1-94, are available. Please contact your Supervisor.

Five attachments are included with this Bulletin: Attachment No. 1 is Bridge Welding Code, AWS D1.5, OSC Contract Administration Guide; Attachment No. 2 is Structural Welding Code, AWS D1.1, OSC Contract Administration Guide; Attachment No. 3 is Safe Code of Practices for Welding Inspection; Attachment No. 4 is Checklist for Compliance with Bridge Welding Code, AWS D1.5, Attachment No. 5 includes copies of Section III "Responsibilities of Caltrans Quality Assurance Person" and Appendix A "Field Inspection Procedure" born the METS Quality Assurance Manual.

Attachments

cc: All Jobs
ACM's
Senior BCE's
RBushey
PStolarski
BGauger

Bridge Welding Code AWS D1.5
OSC Contract Administration Guide

KNOWLEDGE

Review the requirements in the Special Provisions, the Contract Plans, and the Standard Specifications Section 55-l .02, "Welding".

Review the following sections of the Bridge Welding Code AWS D1.5:

- Chapter 3, "Workmanship"
- Chapter 4, "Technique"
- Chapter 5, "Qualification"
- Chapter 6, "Inspection"
- All other sections are to be reviewed as needed.

COORDINATION WITH METS

Prior to meeting with the Contractor at the start of the project, make arrangements to meet with a METS representative to discuss the contract and the welding requirements. This will help establish the roles and responsibilities regarding Caltrans quality assurance inspection.

Refer to the "METS Quality Assurance Manual" for clarification on roles and responsibilities of METS Quality Assurance (QA) personnel. (See Attachment No. 5)

Remember, the Structure Representative has the technical control over all structure work including welding. The Structure Representative is responsible to ensure that the welding is done properly, QA inspection is performed adequately, and the work is fully documented in the project files. METS personnel will provide assistance to OSC for QA inspection of welding work. However, due to limited staffing and difficulties in scheduling statewide METS inspection, OSC personnel must assist METS with QA inspection of welding.

NOTICE TO CONTRACTOR

At the preconstruction conference, schedule a pre-welding meeting with the Contractor. Suggest that they invite to the meeting, the appropriate Sub-Contractors (Welding, Fit up & Erection, Steel Fabricator, etc.) and the Quality Control Welding Inspector(s). Prior to the meeting, provide a copy of the meeting agenda to the contractor and suggest that they bring along their quality control plan, all welders qualification papers, testing information, and any other related welding documents.

Bridge Welding Code AWS Dl.5
OSC Contract Administration Guide

PRE-WELDING MEETING

Conduct the meeting with the Prime Contractor, Sub-Contractor(s), Contractor's Quality Control Welding Inspector and Materials Engineering and Testing Services (METS) representative to discuss the following:

Review specific welding requirements noted on the Contract Plans and Special Provisions.

Section 55-1.02 of the Standard Specifications requires the contractor to submit a "Quality Control Program" listing methods and personnel to satisfy the requirements of Part 6 of AWS Dl.5. Review the Contractor's quality control program (plan) and discuss any deficiency.

The Contractor is responsible for Quality Control (QC) and must appoint a Quality Control Welding Inspector(s). The Contractor's QC Inspector is responsible to review all of the welds and related work performed in the field and to verify that the work is in conformance with the approved Welding Procedure Specifications (WPS) on a continuous basis. Review the qualifications, and responsibilities for the contractors proposed QC Welding Inspectors. (AWS Dl.5, Part 6).

Review the requirements of the Welding Procedure Specifications (WPS), Procedure Qualification Record (PQR), Welder(s), Welding Operator and Tack Welder qualifications. (AWS Dl.5, Part 5, "Qualification"). (METS should cover this portion of discussion).

Suggest the Contractor submits a welding schedule identifying all contract welding work. This will assist field inspection of welds and allow proper lead time for identifying the proper weld test to be performed. Time frames for all test results submittals and-methods of reporting should be addressed.

Discuss AWS D1.5, Section 3.3 "Assembly", to review tolerance and proper positioning of members. Incorrect fit-up will normally result in deficient welds.

Review all documents to be submitted before during and after each portion of welding is performed. (Refer to Attachment No. 2, "Checklist for Compliance with Bridge Welding Code, AWS Dl.5 ")

Discuss corrective measures when welds are not in conformance with the contract documents. (AWS D1.5, section 3.7)

All conversations regarding welding should be documented in writing by the Structure Representative (SR) or SR Assistant. Include a list of all attendees.

Bridge Welding Code AWS **Dl.5**
OSC Contract Administration Guide

FIELD OFFICE FILE

Documents related to welding work are to be filed in the contract's Project Record Files under the following categories:

Category 5 : Copies of all Correspondence
Category 37: Test results for all field tests
Category 41: Certifications for electrodes
Category 42: (New Category) Contractor's submittals (copies of approved Quality Control Program, approved WPS & PQR, welders qualifications, certified welder reports, nondestructive testing (NDT) qualification, certified test reports, calibration reports for NDT equipment, QC Inspector diaries, etc.)
Category 45: Structure Rep. Diaries (including meeting notes)
Category 46: Assistant Structure Rep. And METS diaries

The use of sub-categories should be utilized to keep the welding documents together.

INSPECTION

The Contractors Quality Control Inspector shall meet the inspection personnel qualifications as discussed in AWS D1.5, section 6.1.3.

The welding inspector from METS should be present on the first day of welding, unless other arrangements are made in the pre-welding meeting.

The Contractor's QC Inspector is responsible for all welding operations in the field and is required to monitor the fabrication, set up, root opening, groove angle, equipment settings, and welding papers, etc. on a regular basis. All procedures for welding, testing and documentation must be in accordance with the approved WPS. (AWS D1.5, section 6, "Inspection").

Check the welders welding conditions and verify that the welding tolerances are meeting the contract requirements established in AWS D1.5, Section 3.

The welders, weld operators, and tack welders shall meet the qualifications and use techniques that conform to the approved WPS (consult METS for assistance).

The surface preparation shall be cleaned as necessary to produce sound welds (AWS D1.5, Section 3).

Production rates should be monitored, as needed, to ensure WPS compliance. Production rate fluctuations, especially increases, may indicate non-conformance with the Specifications.

Bridge Welding Code AWS D1.5
OSC Contract Administration Guide

TESTING

Complete joint penetration groove welds in main members shall be QC tested by nondestructive testing. Personnel performing nondestructive testing shall be qualified in accordance with the American Society of Nondestructive Testing's (ASNT) Recommended Practice No. SNT-TC-lA, or equivalent (AWS Dl.5, Section 6.1.3.4). METS personnel will provide assistance for all testing matters.

Radiographic Testing (RT) shall be used for examination of complete joint penetration groove welds in butt joints subject to calculated tension or reversal of stress. See Contract Plans to identify the type of stress in members. If the stresses are not identified on the Contract Plan, contact the Designer.

Ultrasonic Testing shall be used for examination of all complete joint penetration groove welds in T- and comer joints. See Contract Plans to identify the type of stress in members. If the stresses are not identified on the Contract Plan, contact the Designer.

When required, RT and UT may be used to test all complete joint penetration groove welds in butt joints in compression or shear.

Requirements of RT and UT testing are found in AWS D1.5 Section 6.7.1.2.

Weld tabs (extension bars and run off plates) shall be removed prior to testing (AWS D3.12.2.)

Magnetic-particle Testing (MT) shall be used for examination of fillet welds and partial penetration groove welds joining primary components of main members (e.g. web to flange, diaphragm connection plates to web or flange, etc.). Magnetic-particle inspection of fillet welds is not required for secondary members. Consult with designer for verifications.

Requirements of magnetic-particle testing is found in AWS D1.5, section 6.7.2.

Per AWS D1.5 Section 6.5 "Inspection of Work and Records", The Contractors QC Inspector shall keep a record of all WPS qualifications or other tests that are made. The Engineer should get copies of all certified test results and place in the field office files.

Structural Welding Code AWS D1.1
OSC Contract Administration Guide

Similar steps mentioned in Attachment No. 1 for administration of contracts with AWS D1.5 code (Knowledge, Meeting with METS, Notice to Contractor, Pre-Welding Meeting, Filing, and Inspection) should be followed for contracts with AWS D1.1, Structural Welding Code requirements.

The contract requirements for welding, welder qualification, and inspection of welding for projects with AWS Dl.1 Code are similar to the projects with AWS D1.5 Code with the following exceptions:

- Currently there is no contract requirement for the Contractor to submit a "Written Quality Control Program". Future Specifications will be revised to include requirements for a program.
- The qualifications for a qualified engineer or technician proposed by the Contractor as the QC Welding Inspector, need to be "verified" not "accepted", by the Engineer.
- There is no mandatory nondestructive testing required by the AWS D1.1 Code. Welds can be accepted if the Quality Control Inspector and the METS welding inspector find the welding quality to be acceptable by visual inspection.
- When nondestructive testing other than visual inspection is specified in the Special Provisions, it shall be the contractor's responsibility to ensure that all specified welds meet the quality requirements of AWS Dl.l.
- If the Engineer or METS representatives has reason to believe that the welding quality does not meet the specifications, the contractor shall perform any requested testing or shall permit any testing to be performed by the State in conformance with AWS Section 6.

CODE OF SAFE PRACTICES
WELDING INSPECTION

1) All employees exposed to welding and weld inspection work must be trained in the hazards and precautions necessary to conduct the work safely.

2) Electrical Hazards
- stay clear of welding leads, particularly in wet conditions
- inspect leads for frays and missing insulation, all leads must be insulated
- do not touch or remove the ground lead, unless directed to do so by the welder
- welding equipment should be shut off when not in use

3) Fumes and gases.
- Welding operations create harmful fumes and gases, position yourself upwind and away from the welding to avoid exposure
- Do not enter confined spaces where welding is being done, unless properly trained and equipped as required by Caltrans Safety Manual Chapter 14
- Be aware that welding on galvanized or paint coated steel (particularly lead paint) produces toxic fumes and smoke, stay away from these operations unless properly trained and equipped with respiratory protection (See Caltrans Safety Manual Chapter 15)

4) Eye Hazards
- Never look directly or indirectly at welding work, unless you are wearing a welding helmet or goggles with lenses properly shaded for the type of welding being'done (generally a #14 shade is required for large electrodes). Be aware that reflected or indirect arc can also cause eye burns.
- Welding operations should be isolated or shielded to prevent "flash" to adjacent workers or the traveling public.
- Wear ANSI approved safety glasses on the job site to protect from flying particles.

5) Skin Protection

- Stay clear of welding operations. Wear long sleeve shirts or coveralls to protect skin from ultraviolet rays generated from welding.
- Be aware that metal parts may still be hot after welding is done. Wear gloves where appropriate.

6) Radiation

- Weld inspection involves the use of radioactive sources, typically emitting gamma rays. These rays will penetrate clothing and skin, the best protection is to stay away**. Never touch or handle a radioactive source.** Contact the technician or inspection company immediately if you find a source out of it's storage container or unattended.
- The inspection technician must establish an exclusion zone around the work, with warning signs and tape, based on expected and measured radiation emissions**. Do not enter this area unless properly trained and equipped with a radiation detector badge. The** maximum allowable exposure is 2 millirem/hour, but exposures should be kept as low as possible.

Checklist for Compliance with .dge Welding CODE, AWS D1.5

	Item	Reference	If complete ( )
	Identify welded connections as tension, compression, shear or reversal member	Contract Plans	
	Identify all welds (full pen or partial pen)	Contract Plans	
	Identify all members as main or secondary	Contract Plans	
	Review special welding considerations	Special Provisions	
	Review working drawings	Std. Spec. 55-1.02	
	Review contractors written quality control program	Std. Spec. 55-1.02	
**The Engineer Shall verify these items meet the requirements of the project	Review requirements of QC Inspection by the Contractor	D1.5 - 6.1.1.1	
	Review requirements of QA Inspection by the Engineer	D1.5 - 6.1.1.2	
	Engineer and METS approved Welding Procedure Specification (WPS) in file.	D1.5 - 5, Part A	
	WPS Qualification - Gen. Requirements	D1.5 - 5.7	
	Procedure Qualification Record per weld configuration	D1.5 - 5	
	Welding Consumables (Electrodes)	D1.5 - 5.5	
	Electrode certification - SMAW	D1.5 - 4.5.5	
	Electrode certification - FCAW (s or g)	D1.5 - 4.12.3	
	Electrode storage (sticks only)	D1.5 - 4.5	
	Welder, Tack Welder, Welding Operator - General requirements and certification	D1.5 - 5.21	
	Welding positions	D1.5 - 5.22	
	Welder qualification test record	D1.5 - 5	
	Prequalified Standard Joint	D1.5 - Fig 2.4	
	Approved matching filler metals	D1.5 - Table 4.1, 4.2	
	Preheat and Interpass temperature	D1.5 - 4.2	
	Joint Fit-up tolerances	D1.5 - 3.3 Assembly	
	Weld profiles	D1.5 - 3.6	
	Backing bars or back gouging	D1.5 - 3.13	
	WPS for repair work (non-conformance report)	D1.5 - 3.7	
	Weld Termination and Cleaning of weld area	D1.5 - 3.11/3.12	
	Technique / Procedures for SMAW	D1.5 - 4.6	
	Technique / Procedures for FCAW (g or s)	D1.5 - 4.14	
	Inspection of work and records	D1.5 - 6.5	
	Obligations of the Contractor	D1.5 - 6.6	
	Nondestructive testing (NDT)	D1.5 - 6.7	
	All changes to be noted on Contract Plans (As-Built)		

**Note: Most of this information is found in the handout provided by METS.

SECTION III

Responsibilities of Caltrans Quality Assurance Person

1. Primary Responsibility

The primary responsibility of a QA inspector is to insure that the materials and workmanship provided by the contractor meet the requirements of the applicable specifications. The QA inspector is required to verify that all specifications, codes, and special provisions requirements are met and that the contractor's QC reports are in order. The QA inspector shall make random field inspections as a means to accomplish satisfactory QA confidence. Towards the achievement of this objective, the following activities shall be performed on a regular basis :

Schedule daily/weekly meeting times with contractor's QC personnel to monitor job progress and ensure contractors QCP is in effect.

Review all QC reports, weld documentation, and NDT certifications to ensure continued compliance.

Perform random review of radiographs to insure specification compliance. METS assistance at Sacramento is available.

Perform field verification inspections. A minimum of one inspection per location or one inspection per hundred field welds will be desirable.

Document all reported and discovered non-conformance issues and contractor's proposed solutions.

Generate QA inspection reports to be turned over to the engineer in the time frame agreed upon.

To assist the QA inspectors in the performance of the above functions, a ***Field Inspection Procedure*** with the relevant forms has been included in Appendix A . The QA inspector is required to fill out these forms, as applicable, during every field inspection, and disseminate the same to the engineer and all other impacted personnel.

2. Specification Issues.

2.1 It is not the function of the contractor nor the QA personnel to decide issues of materials or design. These issues have been previously decided by the specification, the design requirements, the referenced codes, or will be addressed by the engineering staff as needed. Should a specification problem be brought to your attention, notify the engineer in writing as agreed.

2.2 The contractor must build the product as specified. The QA inspector insures that the product is built as specified by reviewing procedures, qualifications, technique, and documentation. An amount of field verification as determined by the METS Section Chief and the engineer is necessary. Any deviation from the requirements of the specification must be thoroughly investigated and approved in writing by the engineer, prior to implementation.

APPENDIX A

Field Inspection Procedure

For each inspection there will be a check list to assist the inspector in covering all the areas necessary to achieve a complete Quality Assurance program. The following is a list of forms.

1.) Spec. Review: This form will be used to make the Caltrans Engineer and the METS Inspector aware of all required specifications and Testing
A review of the contractors Quality Control and welding paper work.

2.) Prejob: This is a record of the meeting with the Prime contractor, Quality control, the Welding contractor, and Testing company.
A review of the contractors responsibility for Quality Control
A review of the Quality Control Inspectors duties and responsibilities.
A review of Nondestructive testing requirements and the necessary documentation.

3.) Daily Report: This a record of the METS Inspectors Quality assurance on the Job.
A review of the Quality control inspection reports.
A record of any interaction with Caltrans Personnel
A record of any interaction with the Contractors personnel
A record of any verification inspection and nonconformance.

4.) Nonconformance: This is a record of Unacceptable work.
Type of problem (Welding, Fitting, Procedural, etc.)
List of locations
How was the unacceptable work discovered.
Who was notified and when.
What is proposed to rectify the problem.
Each type of nonconformance shall be listed on a separate form.

5.) Notification: This form is documentation and notification to the Caltrans Engineer from the METS inspector that the work does not meet the Specification.
This form will be used when the contractors Q.C. has accepted or overlooked unacceptable work.
A request that the work in question be reinspected.
A request for a written explanation from the Q.C. inspector.

Quality Assurance for field welding

Caltrans Quality Control and specification review

Special Provisions
List all references to welding (Section and Paragraph) and brief description
Highlight all references to other Specifications

1.____

2.____

3.____

4.____

5.____

Standard Specifications
List all references to welding (Section and Paragraph) and brief description
Highlight all references to other Specifications

1.____

2.____

3.____

4.____

AWS (Welding and Nondestructive Testing)
Code: D-1.5 (Bridges)____ D-1.4 (Reinforcing) _____ **D-1.1(Structural)** ___

1.) **Obligations of the Contractor** (Section ___ Paragraph _____

2.) **Inspection Personnel Qualifications** (Section Paragraph_____)
Qualifications of Welding Inspector. (Review documentation)

3.) **Nondestructive Testing** (Section____ Paragraph _____)
Type of test required: RT___ UT____ MT _____
Extent of Testing (Section Paragraph _____)

4.) **Welding Procedure Specifications** (Section Paragraph ____)
(PQR) Procedure Qualification Record ______________________________
(WPS) Welding Procedure Specifications ______________________________

5.) **Welder Qualification** (Section ____ Paragraph ____)

Name	Tests Required	Positions

Quality Assurance for Field Welding

Welding and Inspection Q. C. / Q. A. Meeting

This meeting shall take place prior to the start of the job to assure the contractor will be in conformance with the specifications. **A** representative from the Prime contractor, the approved welding inspector, the Welding contractor, the personnel performing nondestructive testing, the Caltrans Engineer, and the METS inspector should be present.

All parties involved in in inspection or testing shall have copies of all required specification. This should be verified by asking each individual attending the meeting if they have (Special Provisions, Standard Specification, and AWS)

Prime Contractors Representative YES______ NO______
The Welding Inspector Q. C. YES______ NO______
The NDT Inspector YES______ NO______

The role of the **Prime Contractor** in assuring quality work is:

1. The contractor shall be responsible for visual inspection and nondestructive testing. The contractor is responsible for necessary correction of all deficiencies in materials and workmanship. (D1.___ Section ______ Paragraph________)
2. The contractor shall hire qualified and competent personnel to perform inspection and testing
3. The contractor shall schedule nondestructive testing to facilitate attendance by the QA Inspector when requested by the Engineer.

The role of the **Quality Control Inspector** is:

1. Review welding procedures and welder qualification to assure conformance to the specification.
2. Perform inspection prior to assembly, during assembly, during welding and after welding as specified in AWS and additionally as necessary to assure that materials and workmanship conform to the requirements of the contract.
3. The Inspector shall record the locations of inspected areas and the findings of all nondestructive tests, together with detailed descriptions of all repairs made.

The role of the **NDT Technician** is:

1. The Inspector shall identify with a distinguishing mark or adequate document control as approved by the Engineer all parts or joints that the technician tested and approved.
2. The technician shall perform nondestructive Testing in accordance with all applicable Specifications.
3. The technician shall approve satisfactory welds, or reject unsatisfactory welds and report the results to the contractor in writing the same day.

All parties attending meeting :

Print Name	Signature	Date

METS INSPECTOR FIELD REPORT

Contract #______________________ Date______________

Job Name______________________ Report #___________

Company Name, Address_________________________ CWI name __________

Arrival Time________________ Departure Time____________ CWI there?__________

Inspected CWI reports__________

Checked rod ovens____, Electrodes to specification_____, Weld procedures____, Welder Qual____

Joint fitup_____, Mill reports____,

Item Inspected________________ Location_________________________

Summary of discussions with Contractor/Caltrans personnel

Discrepancies noted

Corrections to be made by

Reported to RE

Signature

Qualitv Assurance for field Welding

Nonconformance Report

Contract Number ____________________ Date ______________

Type of Problem: Welding______ Fitt-up______ Procedural______ Other______

Description ____

____

____

Location ____

____

Who found the problem ? ____

____

Who was notified and when ? ____

____

Was the Caltrans Engineer notified ?______________________________________

Name of the Quality Control Inspector ____________________________________

Was the Quality control Inspector aware of the problem ? _________________

What is the Contractor proposing to correct the problem ? ________________

____

____

____

What is METS recommendation ____

____

____

____

Print METS Inspectors Name ____

Title ____________________________Signature______________________________

Qualitv Assurance for Field Welding

Notification of Nonconformance

Contact Number. ____________________ **Date** ____________

Description of nonconformance: ____

____

____

____

Specification reference. Spec. __________ Section ________ Paragraph __________

Draw detail below:
Include dimensions, label areas, and highlight the problem areas with circles or arrows.

The METS Inspector shall notify the Caltrans Engineer of nonconformance as soon as possible. This report is for Caltrans Personnel only and used to help the Engineer assess the problem and the effectiveness of the contractors Quality Control. If it is determined that the Quality Control is not sufficient to assure that the materials and workmanship conform to the requirements of the Specification the Engineer shall request an explanation in writing from the Prime contractor. When unacceptable work is found by the Quality Assurance Inspector (METS) this should be considered evidence that the Quality Control is not sufficient. If it is determined that the Q.C. Inspector is not qualified based on the fact that he does not recognize unacceptable work the Engineer shall have the individual responsible removed and replaced.

METS Inspector ______________________ Signature ____________________

DIVISION OF STRUCTURE CONSTRUCTION ***Bridge Construction Records and Procedures Manual***	B00-08
BRIDGE CONSTRUCTION BULLETIN **Approved:** ____________________ **R. P. SOMMARIVA, Chief** **Division of Structure Construction**	**File: BCM180-2.1** **WELDING** **Date: June 1, 2000** **Expires: None** **Supersedes: BCB 180-2 (12-16-96)**

Subject: Checklist for Welding Quality Control

For all projects advertised after April 1997 (and in some cases by addendum) a section entitled "Welding Quality Control" was added to the Special Provisions as well as a revision to Section 52-1.08 “Reinforcing”. Additional revisions to both sections have occurred since that time.

The Welding Quality Control Section in the Special Provisions, supplements the following sections of the Standard Specifications: Section 49 “Piling,” Section 52 “ Reinforcement,” Section 55 “Steel Structures,” Section 56-1 “Overhead Sign Structures,” Section 75-1.035 “Bridge Joint Restrainer Units,” and Section 86-2.04 “Standards, Steel Pedestals and Post.” Other Sections of the Standard Specifications will be supplemented by the welding quality control plan when required. The welding quality control section also addresses field and shop welding requirements.

Attached are checklists to assist you with understanding the requirements of the welding quality control section contained in the contract Special Provisions. These checklists include a list of applicable contract documents to review and an outline of the responsibilities of the Structure Representative, and personnel from the Division of Materials Engineering Testing Service, Office of Structure Materials (OSM) during each stage of welding. In addition to the checklist, there is a list of commonly used terms and definitions.

The Structure Representative is responsible for all welding. OSM personnel are available to provide advice, guidance, review the welding quality control plan, and perform field and shop welding QA inspection (refer to BCM 180-4 and BCM 180-9 for a list of OSM contacts and phone numbers).

Even though the checklists are extensive, you need to review your Contract Documents for the latest specification requirements.

Copies of the Structural Welding Code-Steel (AWS D1.1) have been assigned to every Area Construction Manager, and copies of the Structural Welding Code, Reinforcing Steel (AWS D1.4), and Bridge Welding Code (AWS D1.5) have been assigned to every Senior Bridge Engineer. **Additionally, the Special Provisions now require the contractor to provide the State, as part of their welding quality control plan, the applicable AWS welding codes for the applicable year noted in the Special Provisions.**

Attachments

C: BCR&P Manual Holders
Consultant Firms
PStolarski, OSM
BPieplow, Construction Program Manager

WELDING QUALITY CONTROL CHECKLIST

DEFINITIONS

The following is a list of definitions commonly used within the section "Welding Quality Control" of the Special Provisions and elsewhere in the contract documents. Additional definitions can be found in ANSI/AWS A3.0-94 "Standard Welding Terms and Definitions."

Certified Welding Inspector (CWI) for State Projects – Inspector certified in accordance with AWS QC1. For State projects the Quality Control Inspector will be a CWI.

FCAW – Flux Cored Arc Welding – An arc welding process utilizing a tubular electrode with the flux contained within the core. The electrode is supplied on a reel and is fed continuously to the welder's gun automatically.

FLUX – A material used to hinder or prevent the formation of oxides and other undesirable substances in molten metal and on solid metal surfaces, and to dissolve or otherwise facilitate the removal of such substances.

GMAW – Gas metal arc welding utilizes a bare or a flux-cored electrode. Gas from an external source is used for shielding. Normally a shop welding process. Often referred to as MIG welding.

Non-Conformance Report (NCR) – A written report originated by OSM which addresses a deficiency being performed and the contract documents not being fulfilled. The report will describe the problem, the location, the Quality Control Inspector response, the proposed solution, and OSM recommendation.

Non-Destructive Testing (NDT) – Testing or an inspection method which does not damage the element being tested (e.g. Radiographic (RT), Ultrasonic (UT), Visual (VT), Magnetic Particle (MT), Liquid Penetrant (PT).

Procedure Qualification Record (PQR) – Documentation indicating testing was performed to qualify a WPS.

Quality Assurance (QA) – This oversight is the prerogative of the Engineer and will be performed by a State representative.

Quality Assurance Inspector (QA Inspector) – The duly designated person who acts for and on behalf of the Engineer. This person is from OSM and will inspect the welding operation and write a welding report for the State.

Quality Control (QC) – Responsibility of the contractor. As a minimum, the Contractor shall perform inspection and testing prior to welding, during welding and after welding as specified in the contract documents and additionally as necessary to ensure that materials and workmanship conform to the requirements of the contract documents.

WELDING QUALITY CONTROL CHECKLIST

Quality Control Inspector (QC Inspector) – The person duly designated by the contractor, to perform inspection, testing, and address welding issues on the project. This person shall be responsible to the contractor for the quality control acceptance or rejection of materials, workmanship, and shall be currently certified as AWS Certified Welding Inspector (CWI) in conformance with the requirements in AWS QC1, "Standard and Guide for Qualification of Welding Inspectors."

Quality Control Manager (QCM) – A representative, employed by the prime contractor, who is responsible directly to the contractor for the quality of all field welding performed. This includes the materials and workmanship. The QCM reviews, approves, and submits all QC documents to the Engineer.

Quality Control Plan (QCP) or Welding Quality Control Plan (WQCP) – A plan submitted by the contractor to the State for each item of welding work to be performed. This plan contains all welding documents required by the contract (refer to the Special Provisions and QCP 1). No welding can begin until this plan is reviewed by OSM and approved by the Structure Representative.

QCP-1, QCP-5 and QCP-7 – These forms are used by OSM and the Structure Representative, as checklists to ensure the contractor's Quality Control Plan or Fracture Control Plan are complete.

SAW – An arc welding process utilizing a solid wire electrode that is fed automatically to the welding head from a reel. A granular flux is automatically deposited from a dispenser onto the molten weld deposit (normally a shop welding process).

Resistance Butt Welding (Flash Butt Welding) – A welding process in which the necessary heat is derived from an arc or a series of arcs established between the bars being welded prior to pressure being applied to join the ends together.

SMAW - Shielded Metal Arc Weld – An arc welding process utilizing a solid electrode with an outer flux coating.

Welding Procedure Specifications (WPS) – A document providing the required welding variables for a specific application to assure repeatability by properly trained welders and welding operators.

Welder's Qualification – Welders must be certified for type and position of weld and weld process. If not certified, tests can be performed to qualify the welders. Welders must be certified and approved by OSM before welding on State projects.

Welding Quality Control Plan (WQCP) – See QCP above.

WELDING QUALITY CONTROL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK

The following contract documents should be reviewed before starting any welding work:

Specific References:

Standard Specifications Sections:
- Section 6-3.02, Testing by Contractor
- Sections 49, 52, 55, 56, 75 and 86 (as applicable to the work)

American Welding Society (AWS) - AWS D-1.1, D-1.4, D-1.5 (appropriate year)
- AWS D-1.1: Prequalification of WPS, Qualification, an Inspection
- AWS D-1.4: Direct Butt Joint Figure 3.2, Workmanship, Technique, Qualifications and Inspection
- AWS D-1.5: Figure 2.4, Workmanship, Technique, Qualification, Inspection, Welded Steel Bridge, and Fracture Control Plan

Contract Plans
Contract Special Provisions
Bridge Construction Records and Procedures Manual
- BCM 9-1.1
- Section 180 – Welding
- BCM 145-16

OSM forms QCP 1 & 5 – (attachment 4)

Before starting any welding, three items need to be completed. A pre-weld meeting with OSM personnel only, a pre-weld meeting with the contractor, and the review and approval of the Contractor's Welding Quality Control Plan (WQCP). These and other items are explained further below.

1. Inform OSM immediately after contract approval that welding, including shop welding, will be performed for your project (see BCM 180-4 & BCM 180-9 for a list of OSM contact phone numbers). At this time, set up an initial meeting with **OSM PERSONNEL <u>ONLY</u>** to discuss the welding requirements for the project, and to plan the pre-welding meeting with the contractor.

2. Conduct a pre-welding meeting with the prime contractor for each type of welding to be performed in the shop or in the field for the contract (i.e. piles, column casings, structural steel, reinforcing steel, miscellaneous metal, etc.). OSM will conduct this meeting if you so request. The Resident Engineer, Structure Representative, Prime contractor, QCM, QC Inspector, any welding subcontractor, suppliers or fabricators and the NDT firm should attend this meeting. The State should have their QA Inspector from OSM present to assist with the following discussion topics:

   a) The submittal and approval process for the WQCP. Supply and explain OSM form QCP-1 (attachment 4) to the contractor. Form QCP-1 is a checklist of the minimum requirements for the WQCP. The contract documents may require additional information to be submitted with the contractor's WQCP beyond those listed on form QCP-1.

PRIOR TO BEGINNING ANY WELDING WORK (Cont.)

b) Discuss the appropriate sections of the AWS code and contract documents as they pertain to the acceptance and approval process of the contractor's WQCP (a OSM welding inspector should cover this portion). The WQCP will be reviewed by OSM and must be approved by the Structure Representative prior to any welding in either the shop or the field. In order for the Structure Representative to accept the WQCP, personnel from OSM will have to review the contractor's WQCP. This acceptance of the WQCP may require OSM personnel to witness the welder(s) welding test plates and the testing of those plates before accepting the WQCP. The same is true of the PQR.

c) Remind the contractor to provide adequate notice prior to starting any welding work (request one week minimum advance notice). This will allow time to schedule an OSM QA Inspector.

d) Inform the prime contractor they are responsible for QC, and they <u>must</u> hire the QC Inspector (a CWI) and the NDT firm, unless stated otherwise in the Special Provisions (i.e. AISC Quality Certification Program, Category Cbr, Major Steel Bridges).

e) Discuss the frequency of inspection, visual and NDT, as well as the frequency of the QCM's submittal of the QC Inspector and NDT reports.

f) Establish a method to identify the welds and lot sizes. This needs to be established for traceability purposes.

g) Discuss the process to randomly select welds to be NDT (see BCM 145-16 for random selection method).

h) Discuss corrective measures when welding does not conform with AWS or the contract documents.

i) Discuss OSM agenda items an any additional requirements addressed in the contract documents.

j) Confirm all discussions of each pre-weld meeting in writing and send a copy to the contractor.

3. Obtain three copies of the contractor's WQCP. Using form QCP-1 and the contract documents, **review the contractor's WQCP and ensure the submittal is complete before forwarding to OSM - this will save review time**. Once the contractor's WQCP is complete, **<u>send two copies</u>** to OSM for their review. OSM will assist the Structure Representative with the acceptance and approval of the WQCP. Keep the other copy in your project files.

4. If resistance butt welding, or any other shop welding is to be performed, ensure OSM has approved the welding procedure, performed a shop audit if required, and performed any testing that may be required to accept the welding process (see BCM 165-10 "Ultimate Butt Splice").

5. After OSM has completed their review, they will notify you by phone, followed by an acceptance memo (QCP–5 for WQCP and or QCP-7 for the Fracture Control Plan, attachment 4) for your project files.

PRIOR TO BEGINNING ANY WELDING WORK (Cont.)

6. After reviewing OSM acceptance memo, and if it is acceptable, place the standard stamp 5-1.02, on both the WQCP and the approved WPS to be used on the project. **Do not place your PE number on the WQCP, the WPS, or the approval letter to the contractor.**

7. Send the contractor a letter approving their WQCP and request 7 copies of these approved documents.

PROJECT RECORDS FILES

WELDING DOCUMENTS ARE TO BE FILED IN CATEGORY 9.

NOTE: To limit duplication and confusion, the Structure Representative may want to use a cross-reference system with the other project record categories (ensuring records can be easily audited). For example, correspondences are filed in Category 5. If the issue is welding, the Structure Representative shall file the document in Category 9 as described in the Construction Manual Section 3-01-2 “Category 5, General Correspondence.”

The following is a suggested list for filing welding documents.

1. The WQCP will be submitted for each item of work for which welding will be performed in the shop and field (i.e. piles, structural steel, rebar, etc.). The WQCP shall conform to the requirements of the Special Provisions and shall include, as a minimum, the items listed on form QCP-1. **Remember, welding is not allowed until the WQCP is accepted by OSM and approved by the Structure Representative.** Each approved copy of the contractor’s WQCP is to be filed in Category 9 "Welding" along with forms QCP-1, QCP-5 and QCP-7 if required.

2. Structure Representatives and their Assistants shall file their reports/diaries in Category 45 and 46 respectively. If welding item work is included within the report, one of two things shall happen: write a separate report, or place a copy of the report in category 9. The welding report shall include: location and type of welding work, amount of production, welder, QC Inspector, QA Inspector, equipment, comments or observations made by either the QC or QA Inspectors and any other pertinent information.

3. OSM welding inspection reports are to be filed in Category 9. All others inspection reports from OSM should be filed in their appropriate Category as outlined in the Construction Manual.

4. If you receive an OSM Non-Conformance Report (NCR) it is to be filed in Category 9, along with the documentation showing what corrective action was taken (repairs and re-inspections of the non-conformance work). **It is the Structure Representative’s responsibility to ensure the non-conformance work is corrected and additional testing and inspection is performed per the contract documents (OSM will only assist in the reinspection when requested and instructed as to the acceptance criteria).**

5. Contractor’s QCM is to submit their welding report to the Engineer within 7 days following the performance of any welding. The Engineer shall review the report for completeness and ensure the welding was found to be satisfactory. The completed report is to be filed in Category 9.

WELDING QUALITY CONTROL CHECKLIST

PROJECT RECORD FILES (CONT.)

6. Copies of all welding correspondences are to be filed in Category 9 (reference Construction Manual 3-01-2).

7. Test results of all field and shop welding are to be filed in Category 9.

8. The contractor shall furnish to the Engineer a Certificate of Compliance for all welding and electrodes used, as required by the contract documents and in accordance with Section 6-1.07 “Certificates of Compliance” of the Standard Specification and Section 8 of the Special Provisions. These certificates shall be filed within Category 9 with reference to the appropriate section within the project files.

DURING WELD PRODUCTION

1. Make arrangements so an OSM welding inspector is present at the job site or the shop on the first day of welding (if the first day is not possible, then the next available day - the key is to provide OSM adequate notice). If welding is being performed in the shop, OSM should be informed by a “Notice of Materials To Be Used” (Form CEM 3101). This will ensure the welding for your project gets off to a good beginning and the QC Inspector has a clear understanding of the QA Inspector’s role and expectations.

2. OSM is responsible for assisting the Structure Representative with QA inspection on the project. OSM is responsible for welding QA at the fabrication shop. Every effort shall be made to ensure a representative from OSM is present during production welding; however, if OSM is not available, the Structure Representative and/or the Assistant Structure Representative shall provide QA inspection and document their findings in their daily reports. The following items should be discussed with an OSM inspector before the pre-weld meeting with the contractor. This discussion should be done in the event a QA Inspector is not available during production welding.

    a) Verify the contractor is providing QC inspection and using the appropriate AWS code, Contract Special Provision, and Standard Specification to evaluate the weld and weld procedure. The contractor is to provide a sufficient number of QC Inspectors to perform the inspection prior to, during, and after welding. The inspection interval of each welder’s work shall not lapse more than 30 minutes, as stipulated in the contract documents.

    b) Verify the welder is listed within the approved WQCP and is qualified and was accepted by OSM to perform the specified weld. For example, if the WPS calls for SMAW in the vertical position, make sure the welder is qualified to perform SMAW in the vertical position. Note: the AWS code specifically disallows a vertical downward progression of welding.

    c) Ensure the welders are following the approved WPS. Items easily verified, include: correct base metals, fit up, joint details (such as bevel angle and root opening), weld process, weld position, electrode type and size, travel speed, voltage and amp settings, preheat and interpass temperature, cleaning/slagging between each weld pass, number of weld passes, and ensuring the welder is placing a string bead and not a weave weld. The QC Inspector should also verify and record these items daily.

WELDING QUALITY CONTROL CHECKLIST

DURING WELD PRODUCTION (CONT.)

d) Review the appropriate AWS code to ensure welding **is not done** when the ambient temperature is too low, when surfaces are wet or exposed to wind, or when welders are exposed to inclement conditions (see the appropriate AWS Code under workmanship or technique).

e) Ensure backing plates, if shown in the WPS, are tight against the base metal or rebar (it might be necessary to grind down only the bar deformations that interferes with the tight fit, not the core area of the reinforcing steel). The Special Provisions, for bar reinforcement, requires the backing plate to be a flat plate. Backing plates are not to be removed for radiographing. If the backing plate is for welding a column casing refer to BCM 180-6 and BCM 180-7.

f) Ensure the electrodes are properly stored. For SMAW electrodes, once the hermetically sealed container is opened, or after electrodes are removed from drying or storage ovens, the electrode exposure to the atmosphere shall not exceed the times stated in the AWS code (typically 4 hours maximum). For FCAW electrodes, they shall be stored in clean and dry conditions at all times.

g) Ensure the welder does not make any errant arc strike (contact between the electrode and the base metal outside the weld area). If an errant strike does occur, the material is subject to rejection, but confer with OSM first.

h) Verify NDT and destructive testing, when required, is being performed properly an in accordance with the Special Provisions and other contract documents (OSM may be of assistance in this regard).

i) Keep an eye on the production and failure rate. A dramatic increase in production and a drop in the failure rate generally result in non-conformance with the WPS.

3. Obtain the QCM welding reports within 7 days or as specified by the contract documents, following performance of any welding. Review this report with the assistance of OSM to determine if the contractor is in conformance with their WQCP. Except for steel piling, this report must be reviewed and a written response approving or rejecting the report must be returned to the contractor within 7 days (your time frame may vary, read your Special Provisions). For piling, this review time will be specified in the Contract Special Provisions.

4. Review all reports regarding NDT, destructive testing, and radiographing. As described in the contract documents, all reports shall have the appropriate signature of the reviewer. For radiographs - the NDT technician, the person performing the review and the QCM shall sign these reports. The reviewers name shall be clearly printed or type written next to their signature. If they are not, return them to the QCM.

5. All radiographic envelopes shall have clearly written on the outside the names of the: QCM, NDT firm, radiographer, date, contract number, complete part description, and include the weld numbers or a report number as detailed in the WQCP. In addition, all innerleaves shall have clearly written on them the part description and include weld numbers, as detailed in the WQCP.

WELDING QUALITY CONTROL CHECKLIST

WELD ACCEPTANCE

1. There are different forms of NDT (VT, UT, RT, MT, and PT) that may be performed on weld elements, but the contractors QC Inspector will always perform a visual inspection (VT) and write up a daily report. The contractor is responsible to ensure all necessary and required NDT is performed. The contractor is also responsible to ensure all welding fulfills the requirement of the contract documents and the appropriate AWS codes. It is the Engineers prerogative to perform QA inspection. If the QC Inspector identifies a defect it is to be noted in the welding report along with the corrective action taken. If the QA Inspector identifies a defect, a Non-Conformance Report will be written and given to the Structure Representative that day**. These reports are not to be given to the contractor. It is the Structure's Representatives responsibility to notify the contractor in writing and ensure the defect is repaired and any additional testing is performed and evaluated. Inform OSM of the repair and request an inspection of the repaired weld.**

2. Welds can be accepted if both the Contractor's QC and OSM QA Inspectors find the welding quality to be acceptable by visual inspection and/or NDT, in accordance with the appropriate AWS code.

3. In addition to the inspection, the contractor shall furnish to the engineer, in accordance with Section 6-1.07 "Certificate of Compliance," of the Standard Specifications and Section 8 of the Special Provisions, a Certificate of Compliance for each item of work for which welding was performed. This certificate shall state that all of the materials and workmanship incorporated in the work, and all required tests and inspections of this work, have been performed in accordance with the details shown on the plans, and the requirements of the Standard Specifications and the Special Provisions.

PROJECT CLOSE OUT

1. The location of all splices need to be show on the as build drawings per BCM 9-1.1.

2. Met with the OSM representative to confirm all NCR and any other details are resolved before accepting the project.

REINFORCING STEEL CHECKLIST

PRIOR TO BEGINNING ANY WELDING WORK

The following items are in addition to those listing in Attachment 2 and are intended to assist you with the inspection of reinforcing steel welding on your project. Therefore, **before starting any welding, review the section entitled "Prior to Beginning any Welding Work" within the "Welding Quality Control Checklist" (attachment 2).**

Specific References:

Standard Specifications Sections:
- 52-1.08B, Butt Welded Splices
- 52-1.08D, Qualifications of Welding and Mechanical Splices

Structural Welding Code - Reinforcing Steel, AWS D1.4 (appropriate year)
- Welding Procedure Qualification
- Welder Qualification
- Direct Butt Joints - Figure 3.2
- Inspection

An approved WQCP is required before any welding is allowed. Review the requirements outlined in form QCP-1, contract documents and AWS D-1.4. AWS D 1.4 does not provide for prequalified welds; therefore, all WPS's and welders must be qualified by testing. You will need a copy of the PQR for each WPS that will be used on the project and the qualification test for the welder(s). The PQR and the welder qualification test must be witnessed by either a lab approved by OSM or by OSM personnel. This should be discussed at the meeting with the OSM representative and also related to the contractor at the pre-welding meeting.

PROJECT RECORDS FILES

In addition to those items listed in Attachment 2, the following items also need to be filed in Category 9 for reinforcing steel welding:

1. The contractor's QCM and OSM QA reports. These reports shall also include the following information when rebar welding and NDT is being performed:

    a) Evidence showing at least 25% of all butt welds were radiographed by the Contractor.

    b) Evidence the Contractor evaluated the results, corrected deficiencies, radiographed repaired welds and radiographed additional welds as required (review your Special Provisions for specific details on additional testing requirements when welds are rejected).

    c) If more than two repairs of any weld are required, the Contractor must submit a repair plan detailing the problem and their proposed solution. This will prevent excessive heat damage to the reinforcing steel in the vicinity of the weld (heat affected zone).

2. A summary sheet recording when radiographs were submitted to and reviewed by OSM personnel and the response to the contractor.

3. Test reports of destructive testing performed for resistance butt welds. OSM will review the testing and perform QA.

REINFORCING STEEL CHECKLIST

DURING WELD PRODUCTION

In addition to Attachment 2, the following items shall also be reviewed by the Structure Representative and/or their Assistant:

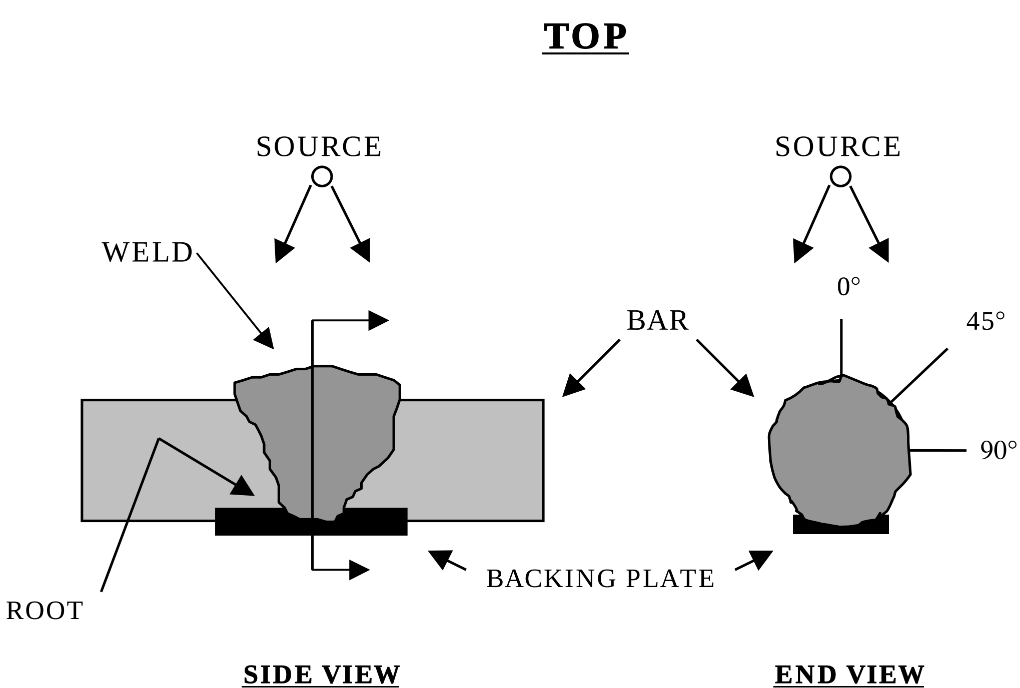
1. Preheat and interpass temperatures. Ensure the proper temperatures are being used for the grade of steel or Carbon Equivalent (CE) being used. Refer to the your Special Provisions, and AWS D-1.4-92 table 5.2 or contact OSM personnel.

2. Bar alignment is within allowable tolerances. For example, AWS D-1.4-92 Section 4.2.3 states, for bars No. 10 or smaller the allowable offset is 1/8 inch. Additionally, Section 52-1.08 of the Standard Specifications states, the deviation in alignment of reinforcing bars at a welded splice shall not be more than 1/4 inch over a 3-1/2 foot length of bar.

3. When specified, a minimum of 6 inches on either side of the welded splice is covered with an insulated wrapping to control the rate of cooling after welding is complete. The method of protecting the weld area from heat loss shall be addressed in the approved WQCP.

4. Randomly select welds to be radiographed (for the random selection process, see BCM 145-16). Verify radiographs are being taken on at least 25% of the randomly selected production lot. If welds or radiographs are rejected, verify additional welds are being radiographed and re-shots of the repaired welds are taken in accordance with the Special Provisions.

5. Verify tests are being performed properly in accordance with the Special Provisions and other contract documents (assistance from OSM is required). Radiographs are to be taken at zero degrees from the top of the weld and perpendicular to the root of the weld as shown below.



REINFORCING STEEL CHECKLIST

WELD ACCEPTANCE

The Contractor should be encouraged to submit radiographs in a timely manner. This will allow the Contractor the opportunity to make corrections if necessary before the work progresses too far. Since the quality of the welding and the radiographing is the responsibility Contractor, the Contractor may choose to continue the work without waiting for OSM review and comment. If so, the contractor proceeds at his own risk and should be informed in writing.

The following items and those in Attachment 2 need to be obtained before accepting any reinforcing steel welding.

1. The contractor shall evaluate the radiographic film and the weld for acceptability and make any necessary repair to the weld and perform additional testing per the contract documents if required.

2. All radiographs, approved, reshot and/or rejected by the QC Inspector, must be reviewed by OSM. When the film is delivered to the Structure Representative, the Structure Representative should prepare a cover memo (attach to the radiograph film) requesting a review by OSM personnel. Before sending your memo and the film, check with your local OSM office for direction and proper sending instructions. On the memo, please include the Structure Representative's name, telephone number and fax number. Each piece of film shall include the contractor's name, date of radiograph, name of NDT firm, initials of the radiographer, contract number, part number, and weld number. The letter "R" and repair number shall be placed directly after the weld number to designate a radiograph of a repaired weld.

3. OSM personnel will review radiographs submitted by the contractor and phone the results to the Structure Representative within seven (7) calendar days after the review or as stated in the contract documents. A written report will follow within 10 working days. To ensure a complete review of the contractor's QC inspection, the radiographs of welds rejected by the contractor's QC Inspector will be reviewed by OSM. OSM will report their findings or the rejections to the Structure Representative as <u>information only</u>. These findings will be reported as either:

   a) "Reviewed film and interpretations submitted by the Contractor are consistent with the Office of Structural Materials findings."

   Or

   b) "Reviewed film and interpretations submitted by the Contractor are inconsistent with the Office of Structural Materials findings and we recommend the Contractor review the QC procedures currently in use."

4. The Structure Representative can accept welds if both the QC Inspector and the QA Inspector agree the welding quality is acceptable by visual inspection and NDT in accordance with AWS D1.4.

REINFORCING STEEL CHECKLIST

WELD ACCEPTANCE (Cont.)

5. In addition to the inspection, the contractor shall furnish to the Engineer, in accordance with Section 6-1.07 “Certificate of Compliance,” of the Standard Specifications and Section 8 of the Special Provisions, a Certificate of Compliance for each item of work for which welding was performed. This certificate shall state that all of the materials and workmanship incorporated in the work, and all required tests and inspections of this work, have been performed in accordance with the details shown on the plans, and the requirements of the Standard Specifications and the Special Provisions.

PROJECT CLOSE OUT

1. The location of all splices need to be show on the as build drawings per BCM 9-1.1.

2. Met with the OSM representative to confirm all NCR and any other details are resolved before accepting the project.

CONTRACTOR'S QCP SUBMITTAL FOR WELDING

To: ______________________, Resident Engineer **Date of this Submittal:** _______________,
Tel. No.: ________________, **Fax No.:** ________________,

From: ___________________________________, **Contract No.:** ________________________,
Welding Firm: ____________________________ **NDT Firm:** ___________________________,

Materials to be Welded: ☐Struct. Steel ☐Misc. ☐Rebar ☐Col. Casings ☐H-Piles ☐Pipe Piles
NDT Required : ☐RT ☐UT ☐MT ☐Visual Only
Specifications : ☐D1.1 (yr) ☐D1.4 (yr) ☐D1.5 (yr) ☐D1.6 (yr)

Our Quality Control Plan for welding to be performed on the subject contract is submitted for your review and approval.
The items checked below are submitted herewith.

QCP ITEMS TO BE SUBMITTED AS A MINIMUM	SUBMITTED	N/A	For R.E. Use
1. Organization Chart showing all QC Personnel & their duties.			
2. Name & Qualifications of Quality Control Manager (QCM).			
3. QC Inspectors - Provide copy of current AWS CWI Certification and eye exam for each Inspector to be used in the work # of submittals____,			
4. Names & Qualifications of Asst. QC Inspectors – Provide current AWS Assoc. CWI Certification or resume. # of submittals_____,			
5. Procedure Manual of NDT firm: certified personnel, NDT equipment, test procedures, calibration methods, methods and frequencies of tests, safety procedures and report forms to be used.			
6. Certifications for Level II NDT Technicians. # of submittals____,			
7. Methods and frequencies of NDT Inspections.			
8. List of Visual Insp. Tools (weld gages, tempstiks, lights, etc.).			
9. Procedures frequencies and extent of Visual Inspections.			
10. Method of tracking and identifying weld joints & welders.			
11. Daily Production & Inspection Log of Welds by QC Inspector.			
12. Action Plan for reporting non-conforming welds.			
13. Prequalified WPS (PQR not required). # of submittals____,			
14. WPS' requiring PQR testing. (PQR tests Must be State Witnessed) # of submittals & tests____,			
15. Standard Weld Repair Procedures.			
16. Electrode & Shielding Gas Certs. for each weld process. # of submittals____,			
17. Welder Qualifications for each process & position that each welder will perform. (Must be State Witnessed) # of submittals____,			
18. Sample Certificate of Compliance form to be used.			
19. One copy each of applicable AWS Welding Codes			
20. Other			

QCM Signature:__________________________ ________________________________ (Printed Name)

METS LETTER OF TRANSMITTAL

REVIEW OF CONTRACTOR'S WELDING QUALITY CONTROL PLAN

To: ___________________________, Resident Engineer
Tel. No.: _____

Date of this Transmittal:
Fax No.:

From: _____, Branch Chief
Date of Receipt (R.E.):_____

Contract No.:
Date of Receipt (METS):

The Contractor's Quality Control Plan Submittal #:_____Rev. # _____ has been reviewed.

☐QCP substantially **complies** with specification requirements and approval is recommended.

☐QCP **needs to be resubmitted** and unacceptable (reject) items corrected as per comments.

(See attached QCP5-NC for Non-Conforming Item Comments)

General Contractor: _____
Welding Firm: _____

Contractor's QCM:
NDT Firm:

Materials to be Welded: ☐Struct. Steel ☐Misc. ☐Rebar ☐Col. Casings ☐H-Piles ☐Pipe Piles
NDT Required : ☐RT ☐UT ☐MT ☐Visual Only
Specifications : ☐D1.1 (yr) ☐D1.4 (yr) ☐D1.5 (yr) ☐D1.6 (yr)

QCP ITEMS REVIEWED	COMPLIES	DOESN' T COMPLY	N/A
1. Organization Chart showing all QC Personnel & their duties			
2. Name & Qualifications of Quality Control Manager (QCM)			
3. QC Inspector-AWS CWI Certification and eye exam for each. # of submittals			
4. ASST QC Inspector - AWS CAWI Certification for each. # of submittals			
5. Procedure Manual of NDT firm: certified personnel, NDT equipment, test procedures, calibration methods, methods and frequencies of tests, safety procedures and report forms to be used.			
6. Certifications for Level II NDT Technicians # of submittals			
7. Methods and frequencies and extent of NDT Inspections			
8. List of Visual Insp. Tools (weld gages, tempstiks, lights, etc.)			
9. Procedures frequencies and extent of Visual Inspections			
10. Describe method of tracking and identifying weld joints and welders production			
11. Daily Production & Inspection Log for Welds for use by QC Inspector			
12. Action Plan for reporting non-conforming welds			
13. Prequalified WPS (PQR not required) # of submittals			
14. WPS' requiring PQR tests. # of submittals			
15 Standard Weld Repair Procedures			
16. Electrode & Shielding Gas Certifications for each process # of submittals			
17. Welder Qualifications # of submittals			
18. Sample Certificate of Compliance form to be used			
19. Other :			

METS REVIEWER:_____
Form METS QCP-5 rev. 4

Date Review Completed:
file: Loc: XX.20. A or B

CONTRACTOR'S FRACTURE CONTROL PLAN (FCP) SUBMITTAL FOR WELDING

To: ___________________________,Resident Engineer **Date of this Transmittal:**
Tel. No.: _____ **Fax No.:**

From: _____, Branch Chief **Contract No.:**
Date of Receipt (R.E.):_____ **Date of Receipt (METS):**

The Contractor's Fracture Control Plan Submittal #:_____Rev. # _____ has been reviewed.

☐FCP substantially **complies** with specification requirements and approval is recommended.
☐FCP **needs to be resubmitted** and unacceptable (reject) items corrected as per comments.
(See attached QCP7-NC for Non-Conforming Item Comments)

General Contractor: _____ **Contractor's QCM:**
Welding Firm: _____ **NDT Firm:**

Specification: D1.5 (yr)

FRACTURE CONTROL PLAN (FCP) ITEMS TO BE SUBMITTED AS A MINIMUM	COMPLIES	DOESN' T COMPLY	N/A
1. Base Metal used meet the project and code requirements			
2. Consumable meet the requirements of heat or lot testing by the manufacturer			
3. Weld metal strength and ductility conform to tables 4.1 or 4.2			
4. Weld metal toughness meets table 12.1 requirements or the undermatching yield strength of a minimum toughness of 25ft-lb @ -20°F.			
5. WPS' requiring PQR test (according to section 12.7). # of submittals:			
6. Prequalified WPS (PQR not required) (According to section 12.7.1) # of submittals:			
7. Base metal repair procedure			
8. Tack weld procedures (According to section 12.13). # of submittals:			
9. Lead QC name, qualifications, and resume. Work history needs to show a minimum of 3 years experience in steel bridge fabrication inspection.			
10. NDT methods, personnel qualifications, eye exams, frequency of testing, reports to be used, and written practice of NDT firm. (see Sec. 12.16.1.2-5)			
11. Electrode & Shielding Gas Certs. for each weld process and base metal combination. # of submittals:			
12. Welder Qualifications for each process & position that each welder will perform. Welder Qualification tests shall be within 6 months of FCM work and shall be qualified by both mechanical (bend) and radiograph tests according to section 12.8.2 and 5 Part B. # of submittals:			
13. Daily Production & Inspection Log of Welds by Lead QC Inspector			
14. Noncritical Repair Welding Procedures such as surface discontinuities. Nocritical repair WPS shall meet the requirements of Section 12.17.2 and 12.17.2.1			
15. Critical Repair Welding Procedures (According to Section 12.17.3).			
16. Other:			

METS REVIEWER:_____ Date Review Completed:

Form METS QCP-7 FCP rev. 4 File Location: XX.20.A or B

OFFICE OF STRUCTURE CONSTRUCTION *Bridge Construction Records and Procedures Manual*	B96-13
BRIDGE CONSTRUCTION BULLETIN **Approved:** ______ **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**File: BCM180-3** **WELDING** **Date: December 16, 1996** **Expires:** **Supersedes: AJ96.14, AJ96.13**

Subject: Reinforcing Steel Welding and Testing

Pursuant to Section 6-3.02, Testing By Contractor, of the Standard Specifications, it is the Contractor's duty to conduct tests in accordance with procedures which are carefully planned, managed and documented by qualified personnel who are specifically designated to perform these functions. In accordance with Section 52-1.11, Payment, of the Standard Specifications full compensation for furnishing and testing sample splices, for radiographic examinations performed by the Contractor, and for furnishing access facilities for inspection and nondestructive testing by the Engineer shall be considered as included in the contract price paid per pound for the bar reinforcing steel involved and no additional compensation will be allowed therefor.

For going projects that have welding and have not received information from the contractor regarding the welding of reinforcing steel, the following procedures should be followed:

- The Resident Engineer is to send a letter to the contractor ordering a submittal of information detailing the contractor's plan to control quality of field welding of bar reinforcement. Use the draft letter in Attachment 1 to this bulletin so that all contractors are treated consistently.

- The Resident Engineer is to review the contractor's submittal for completeness and compliance with the Specifications.

- If the required information is not received within five working days of receipt of a request, the Resident Engineer is directed to temporarily suspend all field welding of bar reinforcement due to the contractor's failure to carry out an order to submit information and failure to demonstrate control of the workmanship, pursuant to Section 8-1.05, Temporary Suspension of Work, of the Standard Specifications.

For projects where the contractor has not started work, the information within Attachment 1 should be received from and discussed with the contractor during the pre-job meeting.

Attachment

cc: StrReps
BCEs
ACMs
OSC HQ Staff
Consultant Firms
Pstolarski, METS
BGauger, Construction Program Manager

From Hither to Yon
13-PAC-00-1.0/10.0
13-123454

June 28, 1996

Contractor, President
Construction Company Name
Company address
Company City, State, Zip Code

Dear Contractor:

Pursuant to Sections 6-3.02 of the Standard Specifications, submit the following, in writing, within five working days of receipt of this order.

1) Contractor's quality control plan for field welding of bar reinforcement as required by Section 52-1.08, "Splicing," and Section B-1.01, "Subcontracting," of the Standard Specifications, American Welding Society (AWS) D1.4, and the special provisions. The minimum requirements are as follows:

   a) List of names and qualifications of the Contractor's Quality control manager and all of the nondestructive testers, welding inspectors, and welders who perform and control the field welding of bar reinforcement work.

   b) Welder qualifications shall include all tests performed to qualify the welders, and verification that the tests were witnessed by the Engineer.

   c) The Contractor's field welding procedure specification for bar reinforcement, including documentation of the tests performed to qualify the specification, and verification that the tests were witnessed by the Engineer.

   d) Description of how the Contractor's quality control plan is updated when any personnel or procedures change.

   e) Description of methods for identifying, tracking, and reporting on; all field welding; quality of the material and workmanship including list of electrode classification, position, type of welding current and polarity; visual inspections; destructive and non-destructive testing; repairing of any deficient welds and reinspection of all repaired welds.

   f) Description of the methods used to ensure the quality control methods are being followed including the frequency of sampling and testing and frequency of reports or submittals to the Contractor.

2) Name of the Contractor's designated person who the Engineer can henceforth consider as the Contractor's quality control manager (QCM). The QCM shall be directly responsible to the Contractor. The QCM shall not be a currently employed subcontractor or currently employed by a subcontractor.

   a) The QCM shall be responsible for administration of the Contractor's quality control plan for field welding of bar reinforcement including but not limited to, the following:

- Review of welding and test records and,
- Assuring compliance with the Contractor's quality control plan for field welding of bar reinforcement and,
- Advising the Contractor and the Engineer of all deficiencies in the welds, tests, inspections and quality control plan for field welding of bar reinforcement compliance.

b) The QCM shall be the individual responsible for submitting and receiving all quality control correspondence and required submittals regarding the field welding of bar reinforcement to and from the Engineer. All such submittals shall be reviewed and signed by the QCM prior to submittal to the Engineer.

. If the plan is not received within the five working day period or if the plan is determined by the Engineer to not satisfy the minimum requirements stated above, field welding of bar reinforcement shall be temporarily suspended, pursuant to Section 8-1.05 of the Standard Specification by reason of the Contractor's failure to carry out an order to submit information and for failure to demonstrate control of the workmanship.

In addition, the QCM shall submit daily log summaries of welding, testing, and inspection for field welding of bar reinforcement to the Engineer at the end of every five working days of field welding of bar reinforcement performed.

Sincerely,

Resident Engineer

ax Prime Contractor, Home Office
Construction Program Manager, Sacramento
District Division Chief - Construction
R.E. File

BRIDGE CONSTRUCTION BULLETIN **Approved: <u>Original Signed by Dolores Valls</u>** **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**File: BCM 180-4** **WELDING** **Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 180-4 Dated August 1, 1997**

Subject: Welding Support from METS

The following link provides contacts within the Office of Materials Engineering and Testing Services (METS) for various welding questions, testing, and/or inspection. Utilize the Structure Material Representative appropriate to your EA or District. In addition to this link, it is recommended that other Structure Representatives who have worked with similar conditions or materials be contacted to ease the burden on lab personnel.
http://www.dot.ca.gov/hq/esc/Translab/smforms/StructuralMaterialsRepresentatives.doc

cc: BCR&P Manual Holders
Consultant Firms
R. Pieplow, HQ Const.
P. Stolarski, METS

BRIDGE CONSTRUCTION BULLETIN

Approved: R Sommariva

R. P. SOMMARIVA, Chief
Office of Structure Construction

File: BCM 180-5
WELDING

Date: December 1, 1996
Expires: None
Supersedes: AJ96.12

Subject: Welding Support from METS for Projects Requiring Radiographs

If you currently have readiographs for a project, send the readiographs to the Sacramento office of Materials Engineering and Testing Services (METS) with a cover letter showing the Contract number, the structure name/number, what part of the structure are the welds located, and a person to contact once the radiographs have been reanalyzed by the Sacramento METS Lab. Send a copy of this memo to the OSC office in Sacramento.

For new radiographs, Sacramento METS (Materials Engineering and Testing Services) has issued guidelines to their employees in the Los Angeles and Emeryville labs. Below is an excerpt from their memo:

1. "Try to attend the pre-construction meeting between the contractor and Caltrans. This will provide the opportunity to go over what is expected during the welding phase of the work.

2. Upon notification by the Structure Representative that welding is ready to begin, visit the jobsite to review the welding procedures, fit-up etc., as well as provide assistance to the jobsite inspector as to what to watch for during the welding operation.

3. If radiographs are to be taken, spot check the bars by the Structure Representative to be shot to make sure they are visually acceptable. This can be done during the day, even if the actual shots will be taken at night.

4. If unable to witness a portion of the actual radiographs being taken, instruct the Structure Representative to have the film delivered to his or her office by the radiographers' company at the start of the next day.

5. Visit the Structure Representatives' office the next day and review the film for the following:

A. Film identification
- -Contract Number
- -Location (bent, column footing, etc.)
- -Weld Number verified by Q.C.
- -Technicians I.D.
- -Make sure information on envelop matches what is in the package

B. Film Qualitv
- -Density (2.5 to 3.5)
- -Check density in weld area
- -Check density in penetrameter

-Check to see if film is free of artifacts

C. Penetrameters

-Size denoted by number
-Placement/Location
-Thickness of shim when required
-Clearly identify hole as required (2T-4T)

After review of the film for items above, send (or have Structure Representative send) the film to Sacramento for review of welds for acceptance or rejection per Office of Structure Construction memo of May 8, 1996 (AJ96.10), (Structural Materials Branch, 5900 Folsom Blvd., Sacramento, CA 95819 ATTN: Paul Hartbower or Frank Reed.)

At the time of the initial review of the film by our field inspectors, the inspector SHOULD NOT accept or reject the weld. However the inspector SHOULD accept or reject the quality of the film.

Many of the problems we have experienced in the past with regard to welding inspection hopefully will be resolved once we get the additional training and staff in place. I feel we need to remember that we are providing a service and our client, in this case, is Structure Construction."

cc: StrReps
BCEs
ACMs
OSC HQ Staff
consultant Staff
RBushey, METS
BGauger, Construction Program Manager

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-6** **WELDING**
Approved: FOR **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: 6/06/97** **Expires: 6/06/98** **Supersedes: None**

Subject: Revise Column Casing Weld Backing Plate Thickness

The attached memorandum (Attachment 1) recommends that the thickness of the weld backing plate for column casings not exceed 3/8". Currently, most design details require backing plates to be the same thickness as the casing up to 1/2" thick. If the contractor requests to use a backing plate with a thickness less than the thickness of the column casing, then it is permitted that an appropriate no cost/no credit Contract Change Order be issued to allow:

- backing plates for column casing to have a thickness equal to 3/8".

Attachments

c: BCR&P Manual Holders
PStolarski, METS
RBushey, METS
ERDavisson, Chief Structure Design
BGauger, HQ Construction Program Manager
Consultant Firms

Business Transportation and Housing Agency

Memorandum

To MR. RICHARD SPRING
MS. DARCY HASSLER
MR. STEVE ELLIS
DMEs - Dist. 1,2,3,6,10,11

Date April 4, 1997

File Policy, Materials, Inspection Guidelines

Fom **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services, MS #5

Subject: Column Casing Inspection/Weld Backing

Currently, a general note in most of the design drawings require "backing plates to be the same thickness as casings up to 1/2" thick" and in some cases a statement that "backing must match casing thickness" with no limit on that thickness.

AWS D1.5-1995 Section 3.13.3 suggests the following backing thickness based on the welding process used: SMAW-3/16", GMAW-1/4", FCAW self shielded-l/4", FCAW gas shielded-3/8" and SAW-3/8".

There is no structural advantage to require backing in excess of 3/8" and fabricators have struggled to break the 1/2"+ backing material sufficiently to allow for proper field fit-up of casing components.

In the interest of a more consistent and better quality product, I recommend that weld backing not exceed 3/8". This should be effective immediately and enforced on all current and future casing projects.

Philip J. Stolarski

PHILIP J. STOLARSKI. Chief
Structural Materials Branch

PJS/mdb

BRIDGE CONSTRUCTION BULLETIN	**File: BCM 180-7** **WELDING**
Approved: R. Sommariva **R. P. SOMMARIVA, Chief** **Office of Structure Construction**	**Date: 7/15/97** **Expires: N/A** **Supersedes: None**

Subject: Inspection of Weld Backing Plate for Column Casing

Attached is a memorandum from Roy Bushey to Ralph Sommariva, referring to the memorandum, dated January 25, 1995 from Richard Spring to METS field inspectors addressing the continuous/non-continuous welding of backing bars used in conjunction with column casings.

After further discussion with METS, the following clarifications were obtained:

> When welding column casing sections together in the manufacturer's shop, the backing bars are to have a continuous full length weld.
>
> The backing bars used to weld the column casing sections together in the field will only be welded to the column casing on one side by non-continuous welds. These welds will be 2" long and at 8" centers. This backing bar does not need a continuous full length weld.

Attachment

c: BCR&P Manual Holders
RBushey, METS
BGauger, HQ Construction Program Manager
Consultant Firms

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. RALPH SOMMARIVA, Chief
Office of Structure Construction

Date: June 6, 1997

File: Policy, Materials, Inspection Guidelines

From: **DEPARTMENT OF TRANSPORTATION**
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services - MS #5

RECEIVED
JUN 1 0 1997
BY

Subject: Column Casing Inspection

It has come to our attention that some of the Structure Construction personnel have questioned the noncontinuous weld on the backing bars used in conjunction with column casings. Back in 1995, the fabricators were having problems with distortion of the edge of the column casings due to the heat generated by the continuous weld. In as much as there was no real need to have the backing bar welded continuously in order to produce a satisfactory field weld, we allowed the fabricator to make the weld as stated in the attached memo dated January 25, 1995.

This memo also addressed some other concerns which were present at that time. The main purpose was to maintain consistency in our overall inspection process.

If you could make this information available to your construction personnel, it may help to relieve some of the questions.

Roy Bushey

ROY BUSHEY, Chief
Office of Materials Engineering and Testing Services

Attachment

cc: PStolarski
RSpring
DHassler
FReed
SEllis
DJones
SMFiles

RJW/llb

State of California

Business, Transportation and Housing Agency

Memorandum

To: MR. HERNANDO MORALES
MR. LARRY WEBSTER
DMEs - Dist. 1,2,3,4,5,6,9,10,11

Datc: January 25, 1995

File: Policy, Materials,
Inspection Guidelines

From : DEPARTMENT OF TRANSPORTATION
ENGINEERING SERVICE CENTER
Office of Materials Engineering and Testing Services

Subject : Column Casing Inspection

Due to some confusion and inconsistency, in the interpretation of column casing fabrication requirements, the following shall become standard inspection policy:

1) Fillet welds on backing bars for field welded joints shall be 2” long on 8” centers. Fillet welds on backing bars for groove welds made in a fabrication shop, where the backing is to remain in place, shall be full length.

2) Intersecting field welds shall be drilled out with a 1” or 2” hole depending on the size of weld at the intersection. No drilling will be required on intersecting shop welds, nor will drilling be required when a field weld intersects an existing shop weld.

3) Exterior casing welds need not be ground flush. However, weld profile must still meet the requirements of AASHTO/AWS D1.5 (Bridge Welding Code), Section 3.6.

This standard inspection policy is to be effective immediately and enforced on all current and future casing projects.

Richard J. Spring

RICHARD J. SPRING
Senior Materials & Research Engineer

RJS/mdb

BRIDGE CONSTRUCTION BULLETIN **Approved: Original Signed by Dolores Valls** **Dolores Valls, Deputy Division Chief** **Offices of Structure Construction**	**File: BCM 180-9** **WELDING** **Date: October 15, 2002** **Expires: July 1, 2003** **Supersedes: BCM 180-9 Dated February 20, 1998**

Subject: Review of Contractor's Quality Control Plan (QCP) for Field Welding

The following link should be used for assistance in reviewing a QCP. Contact the Structure Materials Representative appropriate to your EA or District and they will specify who should receive the plan.
http://www.dot.ca.gov/hq/esc/Translab/smforms/StructuralMaterialsRepresentatives.doc

- Form METS QCP-1; "Contractor's QCP Submittal for Field Welding"

This checklist details necessary requirements of the contractor's QCP submittal and should be made available to the contractor at the pre-construction meeting. The form is intended to facilitate the contractor preparation of their QCP and to ensure a complete submittal plan per the contract requirements. The Structure Representative shall review the Contractor's QCP for completeness prior to sending to METS.

- Form METS QCP-5; "METS Letter of Transmittal"

This form will accompany the Contractor's QCP package when returned to the Engineer. The METS Reviewer will evaluate the Contractor's QCP by checking the elements listed on this form.

Attachment

c: BCR&P Manual Holders
Consultant Firms
P. Stolarski, METS
R. Pieplow, HQ Const.

METS QCP-1 - CONTRACTOR'S QCP SUBMITTAL FOR FIELD WELDING

To: ____, Resident Engineer Date of Submittal:

From: ____________________________________, Contractor ___________________________________,Name of QCM

Contract No.: ____________________________ Welding Firm:

Material to be Welded: ____Piles, ____Reinf. Steel, ____Col. Casings, ____Structural Steel (New & Existing)

NDT Required : ____RT, ____UT,____MT, ____None Name of NDT Firm:____

Specifications : _____AWS D1.1 ( ), _____D1.4 ( ), _____D1.5 ( ) (Indicate Yr. Edition)

Our Quality Control Plan for field welding to be performed on the subject contract is submitted for your review and approval. The items checked below are submitted herewith.

QCP ITEMS TO BE SUBMITTED AS A MINIMUM	SUBMITTED	N /A	For R.E.Use
1. Organization Chart showing all QC Personnel & their duties			
2. Name & Qualifications of QCM			
3. Names of QC Inspectors - Provide copy of CWI Certification for each. No. of submittals ______			
4. Names & Qualifications of Asst. QC Inspectors – Provide Assoc. CWI Cert. or resume. No. of submittals______			
5. Procedures Manual of NDT Firm to be used. List names of certified personnel, NDT equipment, test procedures, calibration methods, typical reports, and safety procedures.			
6. Certifications for Level II NDT Technicians No. of submittals ______			
7. Methods and frequencies of NDT Inspections			
8. Method of tracking and identifying weld joints & welders			
9. Prequalified WPS (PQR not required) No. of submittals____			
10. WPS' requiring PQR tests. No. of submittals & tests_______			
11. Electrode & Shielding Gas Certs. For each weld process and base metal combination No. of submittals______			
12. Welder Certifications for each process & position that each will perform. No. of submittals______			
14. Visual Inspection Procedures (frequencies, gages & tools)			
15. Daily Production & Inspection Log of Welds by QC Inspector			
16. Action Plan for reporting non-conforming welds			
17. Standard Weld Repair Procedures			
18. Certificate of Compliance Form to be used			
19. One copy each of applicable AWS Welding Codes No. furnished ______			
20. Other:			

QCM Signature: ____ ____________________________________

(Printed Name)

METS QCP-5 - METS LETTER OF TRANSMITTAL
REVIEW OF CONTRACTOR'S QUALITY CONTROL PLAN FOR FIELD WELDING

To : ___________________________________, Res. Engineer Date of this Transmittal :_____________
From : ___________________________________ ,Section Chief Contract No. :_____________________

The Contractor's Quality Control Plan submittal has been reviewed.

_____ QCP substantially complies with specification requirements and approval is recommended.

_____ QCP needs to be resubmitted and unacceptable (reject) items corrected as per comments.

General Contractor :_________________________________ Contractor's QCM:_____________________________

Welding Firm :_____________________________________ NDT Firm :___________________________________

Materials to be Welded : ____Struct. Steel ____Misc. Steel ____Rebar ____Col. Casings ____H-Pile ____Pipe Pile

Date of Receipt (R.E.) : ______________ Date of Receipt (METS) :__________________

QCP DOCUMENTS REVIEWED	# OF SUBMITTALS	ACCEPT	REJECT	COMMENTS
1. Organizational Chart - QC Personnel				
2. QC Manager (QCM)				
3. QC Inspectors (Certs. and eye exam)				
4. Asst. QC Inspectors (Certs.)				
5. NDT Firm - Procedure Manual				
6. Weld and Welder ID and Tracking System				
7. WPS/PQR Submittals				
8. Electrode Certs.				
9. Welder Qualifications				
10. Visual Inspection Procedures				
11, Daily Inspection & Production Log (Format)				
12. Non-conformance Report & Action Plan				
13. Standard Weld Repair Procedures				
14. Welding Cert. of Compliance (Format)				
15. Copies of applicable AWS Codes				

NAME OF METS REVIEWER :____ DATE COMPLETED __________________